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REINFORCED CONCRETE ARCH BRIDGE AT PERU, INDIANA.

By Daniel B. Luten.*

One of the longest reinforced concrete bridges ever erected, was completed in December, 1905, across the Wabash River at Peru, Ind. The bridge is remarkable for the short time required for its erection, the contract having been awarded in May, 1905, and the actual time of construction covering less than seven months. Some novel features were introduced also in the balancing of thrusts of arches of unequal spans on extremely light piers.

The bridge consists of seven arches of spans 75, 85, 95, 100, 95, 85 and 75 ft. supported on piers having a thickness of but 6 ft. at the springings and 8 ft. at their bases. The total length of bridge is 700 ft. with a 30-ft. roadway; the height of the roadway above low water is about 30 ft. The contract price was $36,900 for the bridge complete and this proved to be approximately the cost of the structure. The bids for this concrete bridge were lower than those on a competing steel bridge of the same length and roadway and with wooden floor.

All foundations were carried to rock, the excavation varying from 6 to 16 ft. below low water. All concrete was of gravel as it came from the pit without screening, and mixed with cement in the proportions of five gravel to one of cement. The completed structure contains about 5,000 cu. yds. of concrete.

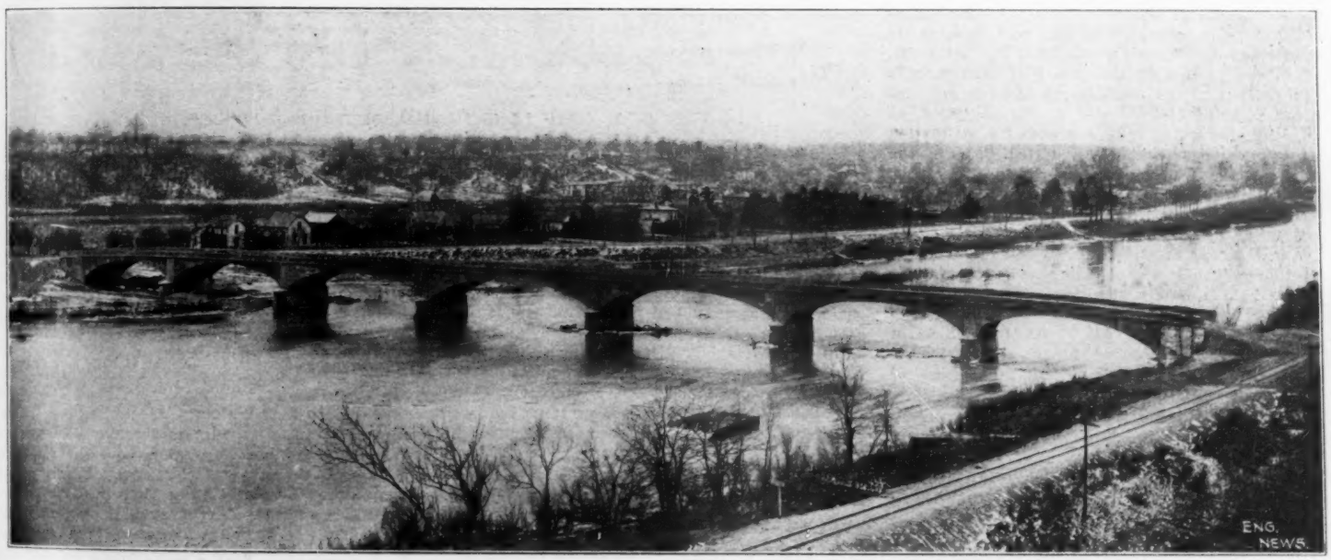
The flood conditions of the Wabash River at this point necessitated a new method of balancing the thrusts between arches of different span. The river has risen at times to an elevation of 3 ft. above the C., C. & L. Ry., whose tracks are shown in the front page picture crossing the north approach to the bridge, and which is the approximate level of the adjoining city streets.

In order to secure as great a waterway as possible, the piers were reduced to miniumu thickness and the spans increased in length towards the middle of the river. This latter feature was considered essential also to a satisfactory appearance of the structure, since a series of arches of equal spans gives the appearance of greatest span nearest the banks, whereas the reverse appearance is greatly to be desired; the middle span should appear to be the longest span and the dominating span of the series. By decreasing the length of span towards the banks moreover the thrusts in the end spans are decreased and the abutments lightened, with consequent economy.

The balancing of unequal spans on light piers has hitherto been accomplished by flattening the shorter arches until the thrust is increased to a point where it equals that of the adjacent longer arch with greater rise. Since the thrust depends on the load as well as upon rise and span, a considerably greater flattening is required than a simple decrease in rise proportional to decrease in span. A span of 100 ft., with a rise of 15 ft., would be balanced by a 75-ft. span having a rise of possibly 6 or 7 ft., which does not give great latitude for increase of span except with heavy grades, or very flat spans. This method of balancing thrusts was used in the Zanesville Y Bridge, Zanesville, O.

For the Wayne St. Bridge at Peru it was impracticable however for the reason that any such flattening of the end arches would have subjected them to complete submergence at times of high water with almost certain collapse of the higher unsubmerged spans. Thus, for example, in the case of the Zanesville Bridge, if the water should ever rise to a point at which the flatter spans would be submerged and their weight thus reduced by the amount of water displaced or nearly 50%, the thrust would be correspondingly decreased and the balancing of the arches upset, and unless the very excessive moments thus produced are provided for by excessive reinforcement, the structure would fall by collapse of the main arches. Such a menace to the Zanesville Bridge is doubtless wholly imaginary, since no such rise in the river at that point is to be anticipated. But in the Wabash River at Peru it was wholly impracticable to keep all spans above high water and at the same time to increase the rise of the central spans to maintain the balance. The grade of the roadway over the bridge is now fully 8 ft. above the grade of the street approach and nearly level from end to end, yet it is but 5 ft. above possible high water level.

The method employed for balancing thrusts in the Wayne St. Bridge, and which is believed to have been used there for the first time, was to incline each arch of shorter span upwards towards the adjacent longer span, so that at any pier the shorter arch has virtually a higher springing than the longer arch, although the springings are maintained at the same actual elevation by slight distortion of the oval curve



REINFORCED CONCRETE ARCH BRIDGE ACROSS THE WABASH RIVER AT PERU, IND.

*President, National Bridge Co., Indianapolis, Ind.

of the shorter span at the pier. A 75-ft. span is thus balanced by an 85-ft. span on a 6-ft. pier, an 85 by a 95, and a 95 by a 100, although the actual rise of the various arches is practically the same, being 15 ft. for the 100-ft. span and about 13 ft. for the 75-ft. span. The effect on the thrusts of the arches by this inclining of the shorter span upwards towards the longer, is shown in Fig. 1 for the pier between the 95 and 100-ft. spans. The thrust T_3 of the 95-ft. arch meets the Thrust T_4 of the 100-ft. arch on that side of the center line of the pier towards the greater span by reason of the elevation of this end of the shorter span. The thrust T_4 of the 100-ft. arch being greater than T_3, the resultant R_4 of the two thrusts is deflected across the center line of the pier and may be kept within the middle third of its base.

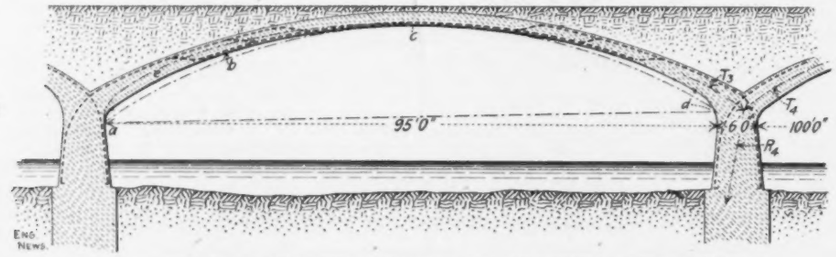


FIG. 1. DIAGRAM SHOWING METHOD OF BALANCING ARCH THRUSTS.

To determine the amount that the shorter span should be raised above the longer adjacent span, the sum of the moments of the horizontal components of T_3 and T_4 about the base of the pier may be equated to zero and their distances above the base varied accordingly. By this method of balancing arches, the inequality of thrust of adjacent arches is counterbalanced by the introduction of a moment in the pier tending to rotate the pier upon its base, and the net result is that a portion of the thrust of each arch is transmitted to the foundations at each pier, instead of being concentrated on the abutment, as in other methods. The approximate inclination of each arch is shown by dotted lines in the half-section of the bridge shown in Fig. 2, the middle span of 100 ft. alone being symmetrical.

The arch rings were reinforced according to the Luten system of single rods extending along the intrados at the crown and along the extrados at the haunches, and crossing the arch ring at points approximately bisecting and trisecting the half-ring. The reinforcement consisted wholly of ¾-in. smooth steel rods of an elastic limit of about 40,000 lbs., spaced 12 ins. apart in the arch ring, constituting less than 2-10 of 1% of the crown section.

The arch curves were designed for linear arches for the earth loading by the approximate method of passing a segment of a circle and a semi-ellipse through three points in common, the crown and the two springings, and then bisecting the vertical ordinates between these two curves to determine the intrados; the extrados is a circular segment. This method has been applied now in several hundred concrete arches of spans up to 100 ft., all of which have proved so universally free from bending moments under earth loading that it was accepted for the design of this bridge without any further analysis of stresses. That it is satisfactory in this instance is evidenced by the fact that with less than 20 lbs. of soft steel to each cubic yard of concrete the settlement of no span exceeded ¼-in. The determination of the intrados curve is shown in Fig. 1, where abcd is the circular segment and aec the curve of the semi-ellipse, the two curves having the points a, c, and d and common, and the intrados is located midway between them.

The process of construction was first to concrete the footings of all piers and abutments, at the same time placing temporary concrete footings in each span to support the centering; the temporary footings were about 2 ft. wide and 12 ins. deep and laid across the 32-ft. width of the bridge at intervals of about 16 ft., so that the weight of 16 ft. by 32 ft. of arch ring was carried by each footing.

The concreting plant was then erected on the south bank, and as span after span of centering was erected continuously from the south towards the north the concreting followed in the order of first the abutment and pier of the 75-ft. span, to their springings, then the arch ring of that span, then the second pier, followed by the 85-ft. arch, and so on. The arches were built in longitudinal rings of about 12 ft. width and continuous from pier to pier. The spandrel walls, together with 3 ft. of arch ring at each spandrel, were not included in this order of concreting, but were deferred until the centers were struck under the springing of each pier, and but one joint in arch ring, at its south skewback. These different steps in the progress of the work are shown by the cross-hatching of Fig. 1.

The concrete was transported from the mixer inside-dump cars of ½ cu. yd. capacity on a temporary track leading over the successive spans and adjustable from side to side of the roadway. From these cars the concrete was dropped to place from heights as great as 30 ft. The reinforcement in the extrados of each arch extends across the ring of the adjacent arch and down the opposite side of the pier and is anchored with a short hook on the end embedded as the work progresses.

The thrust against the two abutments is resisted in part by steel tie rods embedded in a concrete pavement in bed of stream in end spans only, Fig. 1. This pavement serves the additional useful purpose of deflecting the current towards the central spans and away from the banks, thus maintaining the principal channel in the center of the river.

Fig. 3 shows the forms nearing completion for the north 75-ft. span, and the concreting of the piers completed for all spans, and a 25-ft. ring of concrete completed over the five south spans. After this 25-ft. width of arch ring was completed over all spans, the centering that supported this partially completed arch ring was struck in all spans before completing the spandrel rings and spandrels. Three feet of arch ring at each spandrel was built with the spandrel, in order to eliminate the unsightly ragged joint that otherwise would appear at junction of spandrel and ring if spandrel were erected separately upon the ring. The joint between spandrel and arch ring is thus hidden under the arch, instead of appearing in the face—a most desirable result, since the line of extrados can be accentuated successfully only by paneling the forms. By striking the centers under the partially completed rings before adding the spandrels, they were permitted to take their settlement, and thus was eliminated to a large extent the final settlement of the arches which might result in unsightly cracks in the spandrel walls.

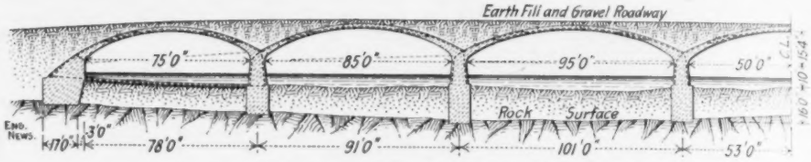


FIG. 2. HALF LONGITUDINAL SECTION OF BRIDGE.

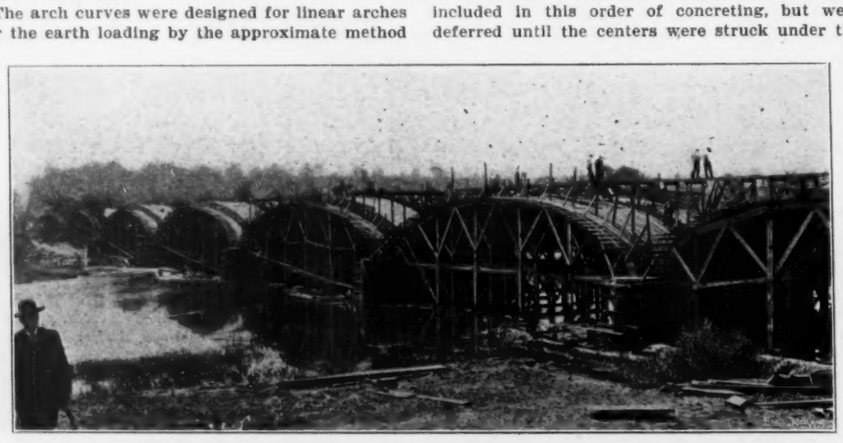


FIG. 3. VIEW SHOWING CENTERING AND CONCRETING.

The centering was designed as described in Engineering News of May 11, 1905, with light flexible uprights held rigidly to place by sway bracing. To strike centers under the 25-ft. rings, the sway bracing was removed from the supports for those rings alone and the uprights allowed to buckle; sufficient sway bracing was then replaced to hold these rings while concreting the spandrels. The sway bracing under the spandrels and spandrel rings was not removed until the centers were finally struck for the entire bridge. Fig. 4 shows the centering under the 95-ft. span after the removal of the sway bracing under the completed rings only and before completing the spandrels. The amount of buckling of the uprights, which are made up of 2 × 12-in. hardwood, is readily seen in this view, and enables an accurate determination of the amount of settlement of these arch rings at that time, about 3-16-in.

The spandrels were concreted continuously over each arch in successive spans from the south end, but with a 5-ft. space omitted at each pier, which was afterwards filled in with a buttress and ice-breaker. An expansion joint is thus provided at each end of the spandrel over each span, and consisting simply of a beveled end abutting against the edge of the post, and with no precautions taken to prevent adhesion save the fact that the post was built against the end of the spandrel after several days of hardening. The spandrel is reinforced with smooth steel rods embedded across the expansion joints and pier middle rings of each arch. The arch rings in each span were bonded together transversely by five ¾-in. rods. In concreting successive spans, the rings of one span were continued across the pier and ended at the skewback of the next span. Thus each span has a transverse joint at the

without any provision for expansion, yet the joints have behaved properly, opening about 1-16-in. at the coping, and with no other cracks showing. The transverse rods in the arch ring were continued into the spandrels, and bent upwards at the end so that they extended vertically through the spandrels to bond the railings. The railing was not built with the spandrel, however, but was added after all centers had been removed from the arches under their full loading of earth filling. The piers are designed with cylindrical ends and conical cap, surmounted at up-stream end by an ice-breaker consisting of a double helicoidal wedge-shaped surface between the conical cap and the spandrel buttress, as shown in Fig. 5. At the down-stream end of the pier the design was the same, except that the ice-breakers were reproduced in a very much reduced form, projecting but 6 ins. from the buttress face.

FIG. 4. VIEW OF CENTERS UNDER 95-FT. ARCH SHOWING BUCKLING OF UPRIGHTS.

As soon as the spandrel posts were completed, the earth filling was placed on the bridge by wagon, and crowned with a gravel roadway. Drainage for roadway and for the earth filling is provided by tiling at each pier leading down through the arch rings.

After completing filling and gravelling, and before building the railings, the material for the railings was distributed the entire length of the bridge, and then the centers were struck and removed from the seven spans, commencing first with the 100-ft. span and following with the two 75-ft. spans, then the two 85-ft. spans, and finally the two 95-ft. spans. Before striking centers and during the filling of the arches with earth, the forms for the railings were erected. The railing forms served as temporary railings while the bridge was hardening, traffic having been admitted to the bridge as soon as the fill was completed.

The total settlement in any one of the spans did not exceed ¼-in., and was quite uniform for all spans, being slightly in excess for the 95-ft. spans. After the bridge had been fully loaded and all centers and falsework removed from beneath all the arches, so that there was no danger of any further settlement of arches or spandrels, the railings were concreted, 36 ins. high and 8 ins. thick, solid with panels, and with expansion joints at piers only, to correspond with the expansion joints in the spandrel. They are reinforced with ¾-in. smooth rods continuously from end to end, and show no cracks except at the expansion joints.

The front page picture shows the completed bridge. About 6,000 bbls. of cement were used for the entire structure: the earth filling on the bridge alone weighs over 10,000 tons, and the superstructure above springings weighs about 16,000 tons with its loading.

The bridge was built by the National Concrete Co. under the patents of the National Bridge Co., both of Indianapolis, Ind. The working drawings, patent right, and all of the steel required for reinforcement were supplied by the National Bridge Co. for the lump sum of $5,640, amounting to about 15% of the cost of the structure.

COMMITTEE REPORTS OF THE AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

Abstracts of some of the principal features of the committee reports presented at the annual meeting in Chicago, March 20 to 22, are given herewith. Particulars as to the other reports and the discussions, etc., are given in our report of the proceedings, on another page.

BALLASTING.

In regard to ballast cross-sections it was the general opinion that some surface drainage was desirable, if not absolutely necessary, even with such material as crushed rock and slag. This requirement is met by the slight rounding of the surface, the top of ballast at end of tie being 1 in. lower than at center and space being left under the rail to allow passage of water. A large body of ballast is necessary around the end of the tie to give support to that part of the material which is directly under the tie. Without this support, the ballast works out and allows the tie to drop. This is met by allowing the ballast to come well up against the end of tie and providing a flat slope from there to the foot. A steep slope, equal to the natural angle of repose of the material, has practically no resistance to a disturbing force, and the ballast should, therefore, be put at such an angle as would resist a disturbing force from the start. Where the ballast contains clay, or material which makes it practically impervious to water, the latter must be carried away on the surface entirely. The importance of a steep slope, in this case, is greater than that of support at the ends of the ties.

DISINTEGRATED GRANITE.—The following notes on disintegrated granite ballast were kindly prepared last spring by Mr. J. B. Berry, at that time Chief Engineer of the Union Pacific Ry.:

In the Rocky Mountains disintegrated granite is usually in mass and very compact, with occasional dikes of hard granite sticking up through it to the surface. The bulk of it may be taken out by a very heavy steam shovel, but we have found it much cheaper to do some blasting. When blasted and picked up, it crumbles into small cubes about the size of peas. The dikes of granite are solid material and have to be blasted out and thrown to one side or used for riprap. We have been able to load with a steam shovel having a 3-yd. dipper about 2,000 cu. yds. per day.

It runs very easily out of the cars, is of a good size to ballast track, is easily handled under all conditions, becomes quite compact when in the track, little or no dust, will stand hard tamping with tamping bar, is one of the best ballasts to shed water, and we find it more economical, especially in renewals of ties, than any other kind of ballast we have used. We have used about a million yards of disintegrated granite for ballast within the last three years that we have definite figures about, and can report as follows:

	Ave. cost per cu. yd. cts.
Cost loaded on cars at pit	9.19
Labor unloading from center dump cars and putting under track	13.37
Tools, engineering and miscellaneous expenses	0.76
Total	23.32

Weight of ballast, 3,000 lb. per cu. yd.

This cost includes everything except haul, and each company must decide for itself what it costs to haul ballast per cubic yard. The items given included tools and repairs of same, powder and fuses, steam drills, wages of pit employees, labor putting ballast under track, repairs to cars, wages of locomotive engineman, fireman and trainmen who spot cars at the pit, roundhouse expenses, water supply, oil and waste for locomotives and steam shovels, fuel, rental of locomotive and steam shovel and rental of outfit cars. In other words, it includes every expense connected with the handling of this ballast except the haul.

While we did not meet with any granite boulders in our gravel pits, we found a good many seams or dikes of the solid granite throughout the pit and were obliged to shoot these. The loading of these after shooting was done by steam shovel and at times was a little slow, but still was the most economical method of handling them. The material that was too large for ballast was used for riprap. The expense of this shooting and loading was included in the figures given for cost of ballast, but the yardage is not included. It probably would be more expensive shooting boulders, as we are able to put holes down the depth of entire face of work. We find it economy to use powder in loosening the face of entire work for the shovel.

J. V. Hanna, C. A. Paquette, C. H. Byers, A. Q. Campbell, M. P. Cotton, L. F. Goodale, G. D. Hicks, B. C. Milner, J. O. Osgood, F. W. Ranno, S. Rockwell, A. F. Rust, G. M. Walker, Jr.

TIES.

An enquiry was made as to the possibility of expressing in a formula the most desirable length and spacing of ties for three types of track: ballasted track for heavy and for light traffic, and unballasted track. It was suggested that the proper factors in such a formula would be the engine axle load, the weight of rail, the allowable bearing pressure of the rail on the tie, and of the tie on the roadbed. Three axle loads, and also three weights of rail were suggested; the weight of rail being intended to accord with the usual present practice as to proportion to the axle loads. The concensus of opinion was that it was impossible to express by formula any rule which would have any value. One member of the committee attempted to determine the proper spacing on the theory that the bending moment in the various rails under their axle loads should be the same.

Fig. 5. View Showing Ice Breakers on Piers.

In view of the increasing size of spikes, the committee recommends that the thickness of tie be made 7 ins.,

and in view of the great scarcity of tie timber in this country (with the corresponding increased cost) that the length of tie should preferably be 8 ft. Consequently it is recommended that the standard tie be 7 ins. by 8 ins. by 8 ft.

For marking ties, it is questionable whether the steel dating nail will give satisfactory service, as attention has been called to the very short life of galvanized nails in some cases, and it would be unfortunate if the nails should fail to have as long a life as the ties. Nothing has developed during the past year to change the conclusions as to the value of treatment. Whatever new evidence has been found has been simply confirmatory of the favorable conclusions of previous years. There has been no large increase in the number of ties treated during the past year. There can be no question but that the more general adoption of treatment would inure largely to the benefit of the railroads. Unfortunately the policy of many roads is to refuse to consider the matter when the price of ties is weak, because of the temporary cheapening of the supply; and to refuse to embark in it when the price of ties is high, because then it is impossible to contract for a supply of ties except at the higher prices.

E. B. Cushing, W. W. Curtis, E. G. Ericson, E. O. Faulkner, C. F. W. Felt, E. E. Hart, V. K. Hendricks, J. C. Nelson, S. M. Rowe, H. R. Safford, H. Von Schrenk.

FENCES AND CATTLEGUARDS.

GALVANIZED WIRE FENCING.—The object of this investigation is to determine the cause of the rapid oxidation or rusting of smooth galvanized wire fences erected in recent years, and, if possible, to suggest a remedy. Fence wire is galvanized to protect it from oxidation, and galvanizing consists in coating the wire with a thin layer of zinc applied by passing the cleansed wire through a zinc bath whose temperature is about 800° F.

The zinc coating or galvanizing on the older barb wire fences, which was galvanized after the barbs were woven in, has withstood oxidation successfully for 18 or 20 years, and there are many examples of galvanized telegraph wire that have been in service for over 30 years and which to-day retain the zinc coating so that the wires are practically as good as new.

All of the smooth galvanized wire woven fences that have been in service for more than five years have lost considerable of their galvanizing, and the exposed iron or steel is rusting. Some strands are good for their entire length, others have lost the zinc coating entirely; the majority of the wires, however, have a fair proportion of the zinc coating intact separated by short stretches having no zinc whatever, and the wire at these points is rusting. These rusted places are found on top, bottom, intermediate wires and stays in such locations as to indicate that its cause is variable thickness, quality or workmanship in the galvanizing rather than on account of the variation in exposure.

As zinc is acted upon by sulphurous acid and sulphuric acid more readily than upon steel or iron, we cannot expect galvanized wire to last longer than bare iron or steel would do when exposed continually to the fumes of switching engines or when located near smelters or other manufacturing plants where much smoke is emitted.

The telegraph companies are not having trouble with the galvanizing of iron wire recently purchased when exposed to ordinary conditions along railways, nor are they having any difficulty in securing sufficient thickness of zinc to withstand four one-minute immersions in a saturated solution of blue vitriol, such as is called for in the galvanizing specification presented by the committee.

The thickness of the zinc coating on wire supplied today for the manufacture of woven wire fence is such as will usually stand only two one-minute immersions in the blue vitriol bath, so it is fair to assume that fence wire galvanizing is only about half as thick as telegraph wire galvanizing. The reason the manufacturers make the fence wire galvanizing so thin is to prevent the scaling of the zinc when the wire is being twisted, knotted and wrapped by the fence weaving machines. The thick layer of zinc is not sufficiently elastic to withstand the work of the machines without scaling, while the very thin coating has appeared to withstand this work better and has therefore been adopted with the very unsatisfactory results above mentioned. Your committee had several samples of fence woven of all No. 9 double galvanized wire on which the zinc coating was of sufficient thickness to withstand the four one-minute immersion tests. Upon examining this fence it was noticed that the zinc on the longitudinal wires did not suffer from the action of the weaving machines; the zinc on the stay wires, however, where they were wrapped around the longitudinal wires, scaled like dry, thick paint would do.

It would appear, therefore, that the best galvanizing protection which we can secure under present methods of fence manufacture is that the longitudinal wires be double galvanized, that is, that they receive a coating that will stand the four one-minute immersions in the blue vitriol bath.

The thickness of the zinc covering for stay wires and fastenings must be regulated to suit the machines on which the various kinds of woven fence are manufactured to ensure against its scaling. This condition applies to all machine-made fence. There are, however, several types of field-erected fence in which both the longitudinal wires, vertical wires and fastenings can be made successfully of double galvanized wire, the cost of which is as much higher as the difference between weaving the fence by machine or having it woven in the field by hand.

One maker, in his desire to increase the life of his fence, coats it by dipping it in white zinc paint, and claims that it "will protect the life of the wire the length of time this coating stays on." The above is only a partial remedy, and if we desire to continue the use of machine-woven fence and secure proper results in the matter of galvanizing a better method would be to use the present thinly galvanized wire to manufacture the fence, and after it is made cleanse it and give it another coating of zinc, as is done in the case of some kinds of poultry netting.

To galvanize fence after being manufactured was suggested to several makers. The flexible joint fence makers objected to this method on account of the soldering effect the zinc would have on the joints. Another maker advises that he has been experimenting in this direction but as yet has not made it a commercial success, the difficulty being in his inability to apply the zinc evenly and thoroughly, together with its excessive cost. One maker of stiff-stay and rigid-joint fence, however, offers to install the necessary plant to galvanize the fence after it is made, provided sufficient business is ensured to warrant the investment.

A thorough investigation of this subject has been made by Dr. A. S. Cushman, of the U. S. Department of Agriculture (Farmers' Bulletin No 239). His contention is that the manganese content, especially if it is unevenly distributed in the steel, is at least in part the cause of the trouble. We are unable either to confirm or disprove this theory, and for the present prefer to recommend good galvanizing as the remedy.

In conclusion; (1) the rapid deterioration of modern woven galvanized fence wire is caused by the coating of zinc being too thin and of an uneven thickness; (2) to procure better protection to the wire and longer lived fence it is necessary to secure an increased uniform thickness of the zinc coating on the wire, and to insure that the galvanizing is intact after the wire has gone through the fence-weaving machines it would seem to require that a second coat of zinc be applied to the fence after it is manufactured.

TENSILE STRENGTH OF FENCE WIRE.—Steel fence wire is manufactured for the trade in three grades: hard wire, medium wire and soft wire. The grades are distinguished by physical tests rather than by chemical composition. Examples of fence wire from several manufacturers were tested with the following results (condensed below: Editor).

	Elastic limit.	Ultimate strength.
No. 9 Soft	830	1,020
No. 10 Hard	1,190	1,500
No. 7 Hard	1,355	1,970
No. 10 Coiled	1,330	1,550
No. 9 Coiled	1,780	2,030
No. 9 Stay	...	1,450
No. 7 Stay	...	2,200
No. 10 Woven	2,090	2,580
No. 7 Woven	2,740	3,520

The best results can be obtained by using hard wire for longitudinals, on account of the increased strength, together with the spring effect of the cooling. Soft or medium wire must be used for verticals in those fences where the vertical wire is wrapped around the longitudinal ones, as well as for knots or fastenings at the intersections of the longitudinal and vertical wires. Different types of fence for the best results require different grades of wire for the verticals and fastenings.

The committee's conclusion is that hard wire, whose ultimate tensile strength for No. 9 gage is from 1,800 to 2,200 lbs., be used for longitudinal wires and that the verticals and fastenings be as near the above in ultimate tensile strength as the action of the various fence weaving or erecting machines will admit without injuring the wire.

W. D. Williams, F. P. Gutelius, F. E. Bissell, S. B. Fisher, F. G. Jonah, A. G. Norton, H. R. Talcott, W. A. Wallace, A. A. Wirth.

BUILDINGS.

ENGINE HOUSES.—The circular form of an engine house is, in general, the preferable design, with possibly the following exception: (A) A through rectangular house, with switches at one end only, may be desirable at branch terminals, or similar points, where not more than three or four locomotives are housed at one time, and where it is more economical to provide a "Y" track than a turntable, or where it is not necessary to turn the locomotives. (B) At shops where a transfer table is used and an engine house is to be added, and at special locations, the transfer table house may be desirable.

SMOKE JACKS.—The design and construction of jacks are at the present time in a state of rapid transition from the adjustable cast-iron form of small diameter to a fixed jack of large size, with a large size flue and a damper, made of various materials. These large jacks were at first made of wood, but are now being made more and more of cast iron, asbestos board and other non-corrosive and non-combustible materials, as fast as manufacturers are able to adjust the construction to the new designs.

The engine house smoke jack should be fixed; the bottom opening should be not less than 42 ins. wide, and long enough to receive the smoke from the stack at its limiting positions, due to the adjustment of the driving wheels to bring the side rods in proper position for repairs. The bottom of the jack should be as low as the engines served will allow, and it should be furnished with a drip trough. The slope upward should be gradual to the flue. The flue for the largest locomotives should be not less than 42 ins. diameter; a damper should be provided in the flue—easily adjusted from the floor. The material used should be non-combustible and non-corrosive.

ROUNDHOUSE DOORS.—(A) Swinging doors are cheap; glass and small doors can be fitted. They are exposed to the wind and liable to be blown against a locomotive, if struck they are liable to serious damage. (B) Rolling doors are made of wood slats or of metal, and small doors on glass are not used in them; they cost about 90 cts. per sq. ft., erected; they are not exposed to damage by wind, and if struck the damaged slats are easily repaired; they fit snugly at sides and bottom. (C) Folding and lifting doors are of wood, and can be fitted with glass and small doors; they are not exposed to damage by wind; if struck they are liable to be seriously damaged. (D) Sliding doors do not fit tightly, and the hangers rust and fail; glass and small doors can be fitted in them; half of the doors are necessarily closed all the time; they are not exposed to damage by wind, but if struck are liable to be seriously damaged.

The tendency to depart from the use of the swinging door, and the introduction of the various kinds of wooden doors of other designs are of recent date, but your committee has made the following conclusions:

Roundhouse doors should be made of non-corrosive material; they should be easily operated, fit snugly, be easily repaired and maintained, and should not be exposed to damage by wind, directly or indirectly, and should admit of the use of small doors. The particular kind of door that will suit a given case can be determined only by giving the proper value to the different factors enumerated above.

A. R. Raymer, E. D. B. Brown, G. F. Bristol, M. Coburn, H. M. Cryder, B. C. Gowen, E. C. Macy, H. M. Steele.

RECORDS, REPORTS AND ACCOUNTS.

ACCOUNTS FOR INDIVIDUAL WORKS.—There is a general agreement among both constructing and maintenance engineers that the record of unit costs is essential information for the guidance of officers who estimate and authorize new work. The objection to any system of individual ledger accounts is based not on principle but rather on the failure of the railway to insure accuracy in the accounts. The general principle underlying the system is certainly correct and since the information derived from these accounts is considered essential, it is worth while to insist on accuracy and to outline an organization of the engineer's office which will bring about the desired result. The objection based on expense is not well taken, because, if the principle is correct, the practice should conform thereto.

An agreement as to principle having been reached, the problem is to outline a system which can be used by all railways at an expense commensurate with the results. According to this system the original data is sent through the various channels to the office of the Assistant Engineer, Engineer Maintenance of Way or Division Engineer, and the clerical force of that office compiles the records and reports and accounts for superior officers. This is the most economical and efficient system.

In connection with any system of individual ledger accounts two special forms are required, the first being known as the Authority Blank, or Requisition for Improvement, etc.; and the second being known as the Ledger.

The information collected indicates that all railways follow a system which requires the engineer to estimate the cost of proposed additions or improvements and to submit this estimate to the executive officer for his approval of authority for the expenditure, in connection with additions, betterments or improvements. The form recommended is practically a requisition for authority to expend money for improvements. It shows in detail the estimate of the cost of the proposed work, as prepared by the Chief Engineer or his representative, and is signed by the executive officer in charge of improvements and is addressed to the officer who will have charge of the work.

The ledger should be of convenient size, of simple design and yet sufficiently comprehensive to give in moderate detail the items of the cost of the work. The book would be composed of loose leaves and the number of

leaves would be in accordance with the special requirements of a particular railway.

RECORDS OF BRIDGES AND CULVERTS.—This report deals with a maintenance office and not a construction office. The two offices may be combined or separated as the case may be, but the system followed in the maintenance office will have features peculiar to itself. The present report has no reference to a system for recording the results of bridge inspections, as this was very carefully considered in former reports. Any complete system of office records of bridges includes two features: (1) A record of the design of bridges. (2) A record of the present condition of bridges as shown by periodical inspections.

The present report deals with the first feature, viz., office records of the design or bridges. The term "Bridges" should be interpreted broadly, to include all structures other than buildings, i. e., structures under the roadbed supporting the traffic. Any office system for recording bridges will apply equally as well to trestles and culverts.

The ideal system for maintenance of way office records of bridges and culverts would be as follows:

(1) The chief engineer, or other officer having in charge the design and care of bridges, trestle and culverts, should keep on file in his office complete plans of every structure showing all its details, its location and the physical characteristics of the territory within a reasonable distance of the structure.

(2) When changes in the design of any structure are made, a record of the changes should be reported to the head office and the record plans should be at once brought up to date.

(3) Division officers, having in charge the maintenance of bridges, should keep on file in their offices, blue prints of the general and detail plans of all bridges, and these prints should be furnished from the office of the chief engineer or other officer in charge of the design.

(4) Photographs of the record tracings of bridges may be prepared and furnished to division or other officers for use in the field. These photographic copies would be of standard sizes and bound in atlas form.

(5) The record of the condition of bridges should be kept in the office.

A careful consideration of the entire subject leads to the conclusion that the system above outlined will insure the most complete, accurate and satisfactory results at the lowest reasonable cost. The proposed system is based on the principle that the chief engineer's office, should, at all times, have on file complete, up-to-date plans of bridges, giving all necessary information for any purpose; and the further principle that division officers require and should keep for their information, copies of the same plans. Incomplete information is frequently very misleading. The man in charge of the care of bridges, wants at times, information in regard to every detail of structure and the only way to obtain this information is from the record plans. The above system also contemplates the use of photographic copies in the field, which can be made at reasonable cost and which will give original and complete information in a convenient form for actual use while on the road. The above system is ideal and complete because it includes an accurate record both of the design and present condition of bridges.

E. F. Wendt, W. S. Kinnear, W. Archer, J. B. Austin, J. G. Bloom, B. T Elmore, R L Huntley, Paul Jones, V D. Simar, J. E. Turk, E. K. Woodward, H. A. Woods.

FREIGHT TRAFFIC OF THE DETROIT RIVER, "SOO," AND WELLAND CANALS.

The domestic freight traffic of the Detroit River has, for the first time, been fully measured, and the results, which have been obtained from the compilation of figures found in reports of masters of vessels filed with the Department of Commerce and Labor through its Bureau of Statistics, and which cover the season of lake navigation of the calendar year 1905 are presented in the following table:

Domestic Freight Traffic of the Detroit River During the Year 1905.

Month	South. Net tons.	North. Net tons.	Total. Net tons.
April	1,575,877	792,711	2,368,588
May	4,551,972	1,352,524	5,904,496
June	5,523,021	1,780,541	7,303,562
July	5,911,625	1,941,534	7,853,159
August	6,300,003	2,314,810	8,614,813
September	4,597,640	1,493,059	6,090,699
October	5,582,089	1,522,905	7,105,594
November	4,593,752	1,578,375	6,172,127
December	1,354,506	871,542	2,226,048
Total	39,991,085	13,648,001	53,639,086

The southbound movement will be seen to be nearly three times as great as that in the opposite direction. This is largely due to the enormous eastbound tonnage of iron ore from upper lake ports to ports along the southern shore of Lake Erie, notably, Cleveland, Fairport, Ashtabula, Erie and Buffalo, whence it is largely shipped by rail to the great iron and steel districts of southwestern Pennsylvania. Were it not for this enormous traffic the through westbound shipments of coal, largely from the same Lake Erie ports, would make the total northbound freight movements through the Detroit River far in excess of those southbound. Of the freight carried in a southerly direction through this river during the past season of lake navigation 1,090,997 tons were flour, 3,176,928 tons grain and flaxseed, 32,900,685 tons ore and minerals (exclusive of coal, of which there was no southbound movement), 1,851,324 tons lumber and 971,151 tons unclassified freight. Of the northbound movement 11,928,158 tons were coal, 6,178 tons grain and flaxseed, 415,533 tons ore and minerals, 11,940 tons lumber and 1,286,192 tons unclassified freight.

In connection with the volume of traffic through the Detroit River, similar information with reference to the "Soo" canals, which connect Lake Superior with the other Great Lakes, and the Welland Canal, which performs the same duty for Lake Ontario, is of interest, and statements are therefore appended showing the amount of freight passing annually through these waterways for ten consecutive seasons.

Freight Traffic Through the Canals at Sault Ste. Marie, Mich., and Ontario.

Season.	South. Net tons.	North. Net tons.	Total. Net tons.
1896	.....	.....	16,239,061
1897	.....	.....	18,982,755
1898	.....	.....	21,234,664
1899	20,619,534	4,636,276	25,255,810
1900	20,532,493	5,110,580	25,643,075
1901	23,087,742	5,315,323	28,403,065
1902	30,275,989	5,685,157	35,961,146
1903	26,932,238	7,742,199	34,674,437
1904	24,213,902	7,332,204	31,546,106
1905	36,778,738	7,491,942	44,270,680

The two canals represented in the foregoing statement are located on opposite sides of the St. Marys River, which connects Lake Superior with Lake Huron, and are owned, respectively, by the United States and Canadian governments. During the season of lake navigation of the calendar year 1905 the total freight movement through the United States canal amounted to 38,802,190 tons, while that through the Canadian canal totaled 5,468,490 tons.

During the lake season of 1905 the difference between the amount of freight carried, respectively, through the Detroit River and the "Soo" canals amounted to 9,368,406 tons in favor of the former route, and this total may be said to represent, with at least a fair degree of accuracy, the traffic through the Straits of Mackinac, as what might be termed the local trade between Lake Superior or Lake Huron and Lake Michigan is comparatively small. The traffic through the Straits of Mackinac consists largely of shipments of grain and flour from Chicago and iron ore from Escanaba, while the westbound cargoes consist largely of coal and package freight.

In the following table, which presents the volume of water-borne traffic between Lake Erie and Lake Ontario, by means of a canal built by the Canadian Government, and which runs parallel with the Niagara River, figures representing freight movements for the canal seasons of 1904 and 1905 are not available.

Through Freight Traffic of the Welland Canal.

Season.	Down. Tons.	Up. Tons.	Total. Tons.
1894	745,942	243,592	989,534
1895	621,928	230,100	852,028
1896	957,928	285,667	1,243,595
1897	1,026,458	218,292	1,244,750
1898	902,519	218,211	1,120,730
1899	622,104	147,514	769,618
1900	579,312	109,245	688,557
1901	501,935	89,301	591,236
1902	567,286	78,811	646,097
1903	715,595	263,212	979,807

BITULITHIC PAVEMENTS may be laid in Chicago, according to a court decision involving an ordinance to improve a street with that material. The court held that the 2-in. wearing surface is only a part of the whole improvement, and that although this is patented yet since the right to use it is open by purchase to all contractors, competition is not barred by specifying this pavement. The North Park Board proposes to replace the ordinary macadam on the Sheridan Drive with bitulithic pavements. Bids for both pieces of work will soon be invited.

STANDARD RULES FOR INFORMATION TO BE SHOWN ON DRAWINGS: PENNSYLVANIA LINES WEST OF PITTSBURG.*

All maps and plans are to be drawn with letters and figures in upright position toward the north and west, the meridian being indicated on the drawing by an arrow.

Maps.—The following should be shown on maps:

(a) Names of rivers, streams, owners or occupants of properties, lot numbers, names of town or city additions or subdivisions, sections, townships, ranges, military or other official surveys, etc.

(b) Official survey stationing, giving station number at each fifth station, with marks for each 100 ft. between; also show location of mile posts.

(c) Dimensions and distances.

(d) All information which is necessary to enable the plan to be used for the purpose intended.

(e) Both magnetic and true meridian, when known.

Plans for Track Changes.—Plans for track changes should have a table showing in feet: (A) Present tracks, (B) New track, (C) Transferred track, (D) Total, and (E) Increase or decrease.

All track plans should show at least one M. P. plus.

All track plans (except ordinary business or industrial sidings) should have a profile, preferably at the bottom, on which grades and the quantities in cuts and fills are marked, and disposition of material indicated; character and amount of bridge and culvert extensions also to be shown.

When scale plans for new passing sidings, running tracks, or additional main tracks are made, the plan should be accompanied by standard track diagram and profile showing the proposed full scheme of which the plan submitted is to be a part.

Plans for engine house and ash pit layouts must have profiles showing grades of tracks, and elevations of ash pit, turntable and engine house floor.

Signal and Interlocking Plans.—Signal plans should show clearly:

(a) All switches to be included in interlocking plant.

(b) Section and weight of rail within the limits of work.

(c) Distance between track center lines, with notes whether tracks can be spread to allow location of high signals between them.

(d) Direction and class of traffic on each track.

(e) Location of signal tower with respect to work, and nearest station or mile post.

(f) Location or lead out.

(g) Location of telegraph or telephone pole line where electric circuits are involved.

Bridge Masonry.—Bridge masonry drawings should show clearly:

(a) A small scale general location plan with alinement of tracks, location of stream or road several hundred feet above and below the bridge, and distance along the center line of track from face of abutment under coping to the nearest official profile station number.

(b) A small scale profile of grades over and near bridge, with elevation of highest known water marked thereon.

(c) Lines of standard roadbed and ballast section.

(d) Character of foundation material and construction of foundation work.

(e) Elevation above ocean datum of foundation, high water, bridge seats, and base of rail (low rail on curves).

(f) Estimates of quantity of excavation, quantity of masonry, piles, timber; pressure intensity on foundation, material, etc.

Profiles.—Profiles should be drawn with right-hand end toward Pittsburg and should show clearly:

(a) Ocean datum line.

(b) Datum lines for cities, U. S. Government, etc., referenced to ocean datum.

(c) Grade line (base of rail—low rail on curves).

(d) Line of top of slope in cuts and toe of slope on fill, on each side of roadway.

(e) Elevation of high and low water of adjacent streams and rivers.

(f) Description and elevation of bench marks.

(g) A sketch of track alinement near bottom of sheet, with arcs of circles to show direction of curves.

(h) Stationing and elevations at important places.

General.—The number of the notebook containing the field notes should be shown on map or plan.

If a map or plan is to be anything more than a picture, it must have the dimensions and distances marked on it.

Plans requiring the approval of officers should have proper place provided for that purpose, preferably near the title.

*Accompanying the report of the Committee on Records, Reports and Accounts, presented at the annual meeting of the American Railway Engineering and Maintenance of Way Association at Chicago, March 20, 22, 1906.

A 10,000-HP. SINGLE-WHEEL TURBINE AT SNOQUALMIE FALLS, WASH.

By Arthur Giesler.*

For some months the Seattle & Tacoma Power Co. has had in operation at its Snoqualmie Falls plant, 35 miles east of Seattle, Wash., a single-wheel turbine of 10,000-HP. capacity. It is the largest turbine of its type ever built. The 12,500-HP. vertical turbines of the Electrical Development Co., the 10,000-HP. horizontal turbines of the Ontario Power Co., and the 10,000-HP. vertical turbines of the Canadian Niagara Power Co., all of which are at Niagara Falls, Ont., are duplex machines, as each unit has two runners on a single shaft driving a single generator. The Snoqualmie Falls turbine, with but one wheel, therefore represents by far the largest concentration of power yet accomplished in turbine water wheels.

The turbine in question represents an enlargement of the Snoqualmie Falls plant to double its original capacity. The plant† utilizes the power of the Great Falls of the Snoqualmie River, a mountain stream flowing down the western slope of the Cascade Mountains, and transmits electric current to Seattle (25 miles west on an airline) and Tacoma (35 miles southwest). It was built in 1898 and 1899, and delivered its first current for commercial service in the middle of the latter year. The fall is about 270 ft. high, and gives an available power, at minimum flow, of 30,000 HP., though by means of storage upstream the available power can be increased to several times this figure. The initial installation of machinery was of 10,000 HP. capacity, but the head-bay and the tailrace tunnel were built for 20,000 HP., the plan being to extend the power-house and put in additional machinery to raise the capacity to this figure as soon as the demand for more power was in sight.

The Snoqualmie Falls power plant is unique among hydraulic power developments. It employs neither the common arrangement wherein the power-house is located on the river bank below the falls, nor the wheelpit construction as used in four plants at Niagara Falls, but an arrangement intermediate between the two. The great cost of the wheelpit made it inadmissible, and on the other hand the heavy spray and mist below the falls threatened to interfere with the proper operation of electrical machinery if the power-house were located on the river bank. For these reasons the machinery is placed in an underground chamber excavated in the solid basalt rock some 250 ft. below the surface. This chamber forms the power-house. Its dimensions are, roughly, 200 ft. by 40 ft., by 30 ft. height; its longer axis is parallel to the river, and the lower end is about 300 ft. upstream from the crest of the falls. From its downstream end a tailrace tunnel leads to the lower reach of the river. At the upper end of the chamber a vertical shaft, 10 × 27 ft. in section, rises to the surface; the middle of this shaft is occupied by an elevator, and the end compartments contain the two 7½-ft. penstocks.

Every part of the plant except the power chamber and the machinery was built for the full development. To extend the installation of machinery, when a demand for the second 10,000 HP. would be in sight, it was planned to excavate a second chamber equal to the first and on the same axis, extending upstream from the shaft.

Before the turbine was installed, the main power equipment of the station comprised four 1,500-KW. units, each composed of a [illegible]-wheel Doble tangential water-wheel of 2,500-[illegible] capacity and a 1,500-KW. Westinghouse three-phase alternator. These units run at a speed of 300 r. p. m., and produce 60-cycle current at [illegible] volts pressure, which is raised to 30,000–[illegible] volts for transmission. The water-wheel units were arranged with two housings each, [illegible] 45-in. wheels in a housing, and two 3⅞-in. [illegible] per wheel.

About two years ago it became evident that increased generating capacity would soon be required. A small amount of space at the end of the existing power chamber was available, but not enough however to accommodate much power if the old type and size of units were employed. It appeared that by using a turbine unit of large capacity this space would suffice, without additional excavation, for a unit developing a large part of the capacity of the second penstock. On detailed inquiry, including proposals and bids from the leading water-wheel builders in this country and abroad, it was found that a single-wheel turbine could be obtained, within the limits of the available space, that would develop the full 10,000 HP. This machine was proposed by the Platt Iron Works Co. (formerly the Stillwell-Bierce & Smith-Vaile Co.) of Dayton, O. It was designed for an effective head of 260 ft. and a speed of 300 r. p. m., the latter fixed by the limitations of generator design. While slow-speed generators of the capacity of the proposed unit were in existence, and smaller generators running at speeds equal to or higher than 300 r. p. m. had also been built, there was no precedent for a machine approximating 10,000 HP. capacity running at a speed anywhere near as high as that proposed. The Westinghouse Electric & Mfg. Co., of Pittsburg, Pa., however, undertook to build such a generator. The contract for the turbine was let on the proposal referred to, and the machine was built and installed and is now in successful operation.

For the original installation a generating unit of this size would doubtless have been quite out of the question, because of the need for flexibility and reserve. In enlarging the station, however, the four existing units ensured ample flexibility as regards handling the output, irrespective of the size or number of new units. The advantages of the large-capacity unit in economy of space and cost and in efficiency of operation did not, therefore, entail the disadvantages of excessive concentration of generating capacity.

The drawings shown herewith exhibit the construction of this large turbine. Figs. 1 to 4, giving respectively a longitudinal section, plan and elevations of the turbine, show the general disposition of parts. The turbine is a horizontal-shaft machine, and is of the highly-popular Francis type, i. e., radial inward flow, with central axial discharge. The turbine proper has only one shaft-bearing, which is located on the side away from the generator; the generator having two bearings makes the complete unit a three-bearing machine. This arrangement has proved to be eminently satisfactory and is considered a feature of the general design. The discharge-pipe or draft-tube of the turbine is on the generator side (front side). The shaft-bearing, thrust-bearing and thrust-balancing devices are located at the back side. The wheel is 66 ins. outside diameter by 9½ ins. wide through the vanes. It has 34 vanes, which extend a short distance inward beyond the end plate of the wheel on the discharge side, thus giving a slight "axial" effect and making the wheel a turbine of "mixed-flow" type rather than a pure radial-flow turbine. Suitable axial curvature is given to the tail of the vane to produce this axial reaction effect, as may be seen from Fig. 5. The guide-vanes, 32 in number, are of the swivel or Fink type. They are connected by means of arms projecting radially inward (see Fig. 3), to a rotatable ring, concentric with the turbine shaft; this ring is actuated through connecting-rod and shaft by a Lombard governor, whose regulator is driven from the generator coupling as is clearly shown in the drawing.

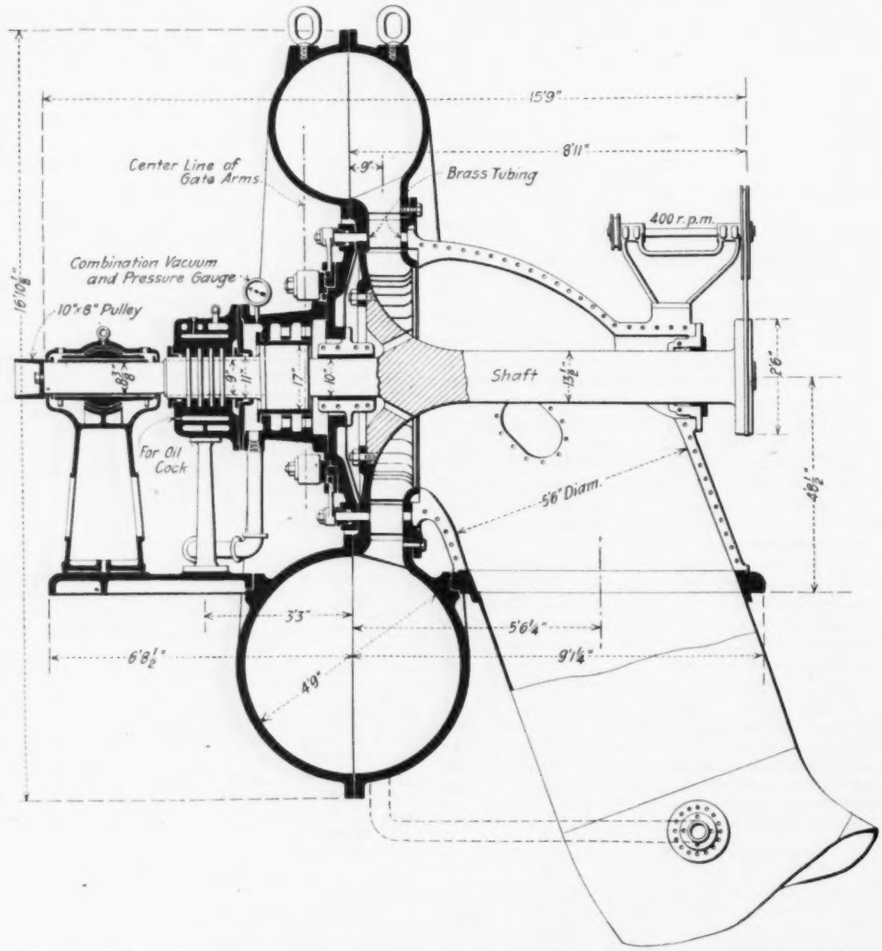


FIG. 1. LONGITUDINAL SECTION OF 10,000-HP. SINGLE-WHEEL HORIZONTAL TURBINE AT SNOQUALMIE FALLS POWER PLANT OF SEATTLE & TACOMA POWER CO., SNOQUALMIE FALLS, WASH.
Effective Head, 260 ft.; Speed, 300 r. p. m.

*Consulting Engineer, 170 Broadway, New York City.
†See Engineering News of Dec. 13, 1900, p. 398, for a full description of the plant as originally built.

The housing of the turbine is built of cast-iron segments, whose size was dictated by the fact that the turbine parts had to be lowered down the elevator shaft, the only means of access to the power chamber. The radial joint-face of the housing, which divides the spiral water case into halves, lies to one side of the vane-passage, as can be seen in the longitudinal section, Fig. 1. This disposition was adopted to secure greater lateral rigidity of the housing at its inner edge. In turbines with other types of regulating mechanism the fixed guide-vanes tie the housing firmly together; with swiveling guide vanes, as in this turbine, such tying or stiffening effect is lacking, and at the same time greater stiffness than with fixed vanes is required, in order that the vanes may swivel freely, without binding. For this reason a set of tie-diaphragms or spacers, which also act as preliminary guide vanes, is provided just outside of the swivel vanes, as clearly shown in Figs. 1 and 4. The arrangement of the radial joint one side of the axis of the turbine wheel results in keeping these spacers clear of all joints, making it possible to cast them integral with the sections forming the sides of the vane passage. In order to secure ample strength in these spacers without making them so thick or numerous as to interfere perceptibly with the free flow of water, a 2-in. rivet is driven in a hole drilled longitudinally through each, from outside to outside of housing.

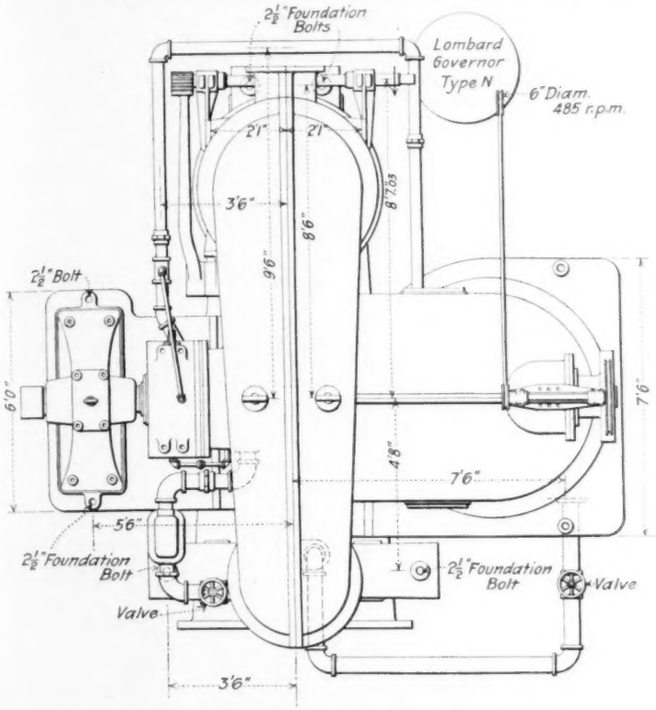


FIG. 2. PLAN OF 10,000-HP. SNOQUALMIE FALLS TURBINE.

The weight of the stationary parts of the turbine is carried directly by the housing; the latter rests on the foundation by three spread feet, two at the inlet and one at the governor end, cast integral with the base section of housing. The weight of the moving parts, carried by the shaft, rests on the rear water-wheel bearing and on the rear generator bearing. Where the shaft passes through the discharge casing it is packed by a stuffing-box, which carries no vertical load. The rear water-wheel bearing is a pedestal which foots on the foundation independently of the main housing; it is tied laterally to the housing by a floor-plate at the foundation level. The bearing is a self-alining spherical bearing 8⅜ × 26 ins., and has two oiling rings.

The turbine wheel or runner is an annular steel casting whose radial depth is only enough to contain the vanes. There is no hub or spider, but the shaft is enlarged into a disc of sufficient diameter (46 ins.) to permit of bolting the wheel ring directly to it. The vanes of the runner are finished by filing smooth. The shaft is a nickel-steel forging made by the Bethlehem Steel Co.; it is 13½ ins. in diameter at the generator end, with a 30-in. coupling flange, and 8⅜ ins. in diameter at the rear end; its total length is 15 ft. 9 ins.

The means employed for balancing the thrust of the runner are of special interest. Single-wheel horizontal-shaft units are relatively infrequent in turbine practice, especially in large sizes, where the thrust of a single runner is large enough to require careful consideration. The thrust is made up of two parts: (1) that due to the static pressure or effective head of water at the various points of the runner surface; and (2) that due to the deflection of the water from a purely radial path through the wheel. As concerns the first part, the front face of the wheel is pressed upon by a pressure varying from the supply head at the outer circumference to the discharge pressure (vacuum) at the inner edge of the vanes, which latter extends over the whole central area of the runner (and shaft extension). The rear face of the runner is subjected to the pressure of water leaking through the radial air-gap between casing and runner, substantially equal to the supply head. This greatly over-balances the pressure on the front face, and the resultant thrust is to the right in Fig. 1 (toward the draft-tube). The discharge ends of the vanes, being curved transversely, also have a pressure component directed toward the right. The yel-

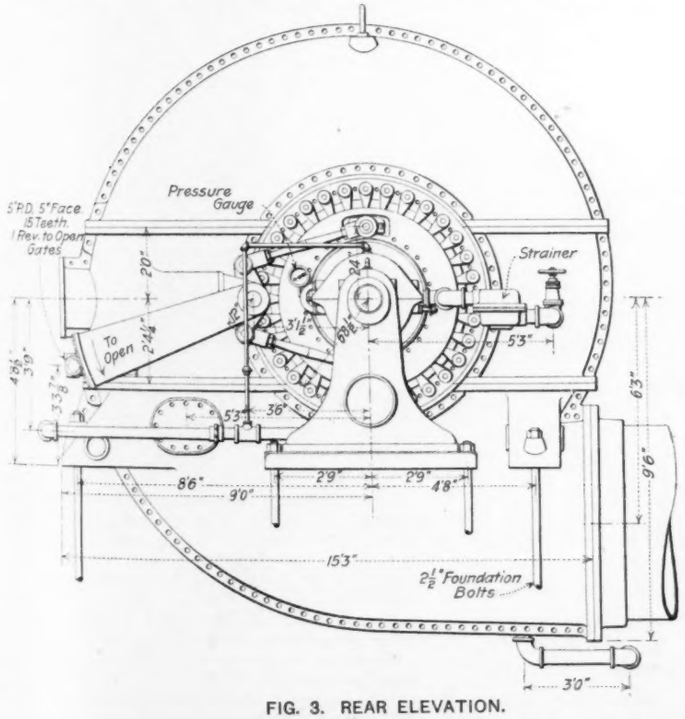


FIG. 3. REAR ELEVATION.

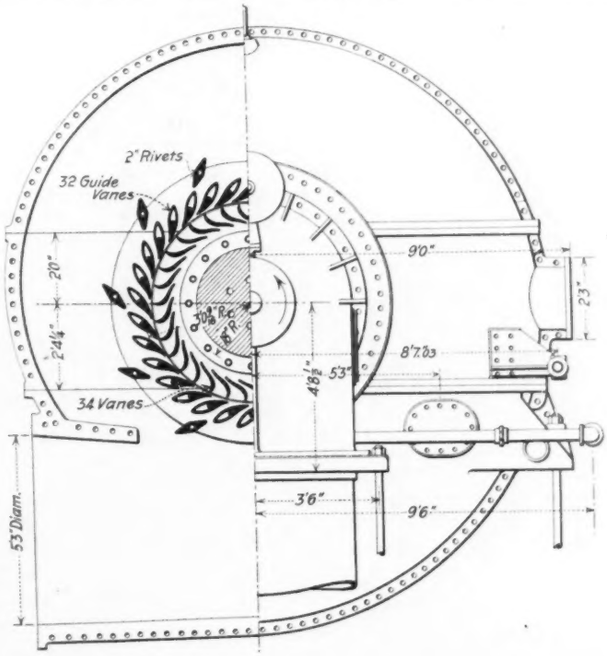


FIG 4. TRANSVERSE SECTION AND FRONT ELEVATION.

ocity effect produces a thrust directed toward the left, but this is very small and does not materially reduce the pressure thrust.

By far the larger part of the pressure thrust is eliminated by venting the space back of the runner into the discharge space. Six holes through the wheel near the shaft, indicated in Fig. 1, have this function. The water leaking in through the air-gap is continuously discharged through these vents into the draft-tube, and the accumulation of any large static pressure back of the wheel is thereby avoided. There is a certain amount of centrifugal action, however, in this back space, due to the water being in part carried with the wheel, which results in producing a pressure toward the periphery, reducing the relieving action of the vents. To minimize this effect, the back head of the wheel case bears on its inner face a number of radial ribs projecting as close to the back face of the wheel as mechanical considerations permit; these ribs act to hold the water from rotating with the wheel and correspondingly reduce the centrifugal effect.

By these means the average pressure back of the runner is reduced to near atmospheric pressure, being slightly above atmosphere with full gate opening and slightly below atmosphere at low gate opening. The average pressure on the front of the runner, however, is always lower, and the resultant thrust is therefore toward the draft-tube, though its amount varies considerably, being greatest for full gate opening. This thrust is taken up by the balancing piston immediately back of the rear head of the wheel case, and the ultimate balance and adjustment of position is accomplished by the collar thrust bearing behind the balancing piston.

The balancing piston is a forged enlargement of the shaft, finished to a diameter of 17 ins., which works in a brass sleeve set in a hub-like projection on the back of the wheel-housing. The inside of the sleeve has six circumferential grooves, each 1-in. wide and ¼-in. deep, as water packing. The chamber in front of the piston communicates by a pipe (containing a strainer) with the supply casing of the water-wheel and therefore receives the full pressure of the supply head. The chamber back of the piston is drained to the draft-tube, so as to carry off any leakage past the piston. The device thus produces a constant thrust on the piston, directed toward the left. By throttling the pressure pipe this thrust can be adjusted as desired.

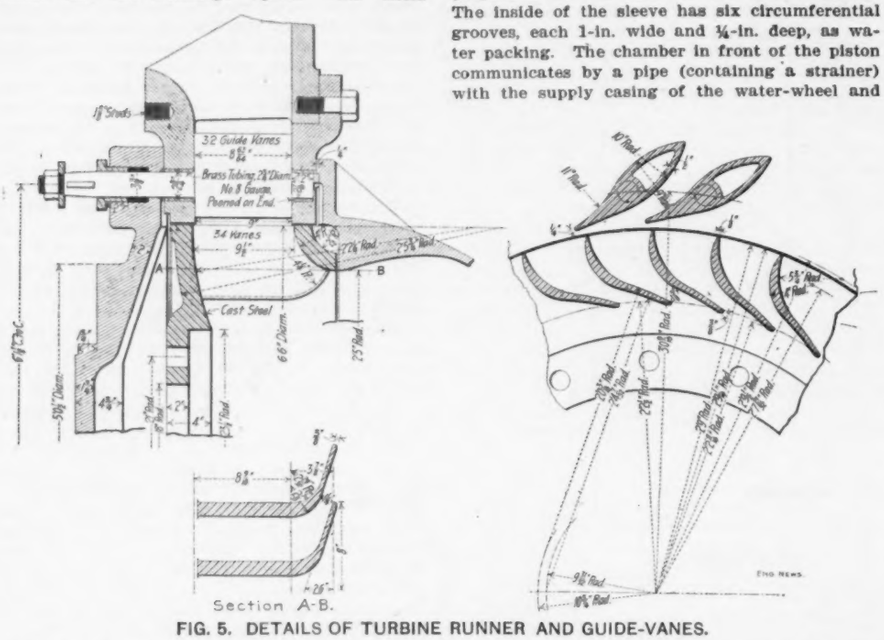


FIG. 5. DETAILS OF TURBINE RUNNER AND GUIDE-VANES.

The thrust bearing, shown in Fig. 1 and in detail in Fig. 6, consists of a group of four collars on the shaft, working in a babbitted thrust-block which is bolted to the back of the wheel-housing. The collars are formed on a steel sleeve which fits over the shaft and is bolted to the rear face of the balancing piston; this makes it possible, when the collars are worn out, to renew the bearing by dismounting the thrust block and placing a new sleeve. The thrust bearing is lubricated by oil immersion. An oil chamber is cored in the block, and communicates by numerous oil holes with the bearing faces; a constant flow of oil is maintained by means of oil supply and drain pipes. Concentric with the oil chamber and outside of it a water chamber is cored in the block. Cooling water is supplied to this chamber by a pipe from the pressure side of the turbine, and drains from the top of the bearing through a drain pipe to the draft tube. A U-pipe attached at one side of the bearing forms connection between the water chambers of the upper and lower halves of the block. This detail avoids making the connection by a hole through the joint face, which would allow leakage of water into the oil-space and [illegible] the bearing.

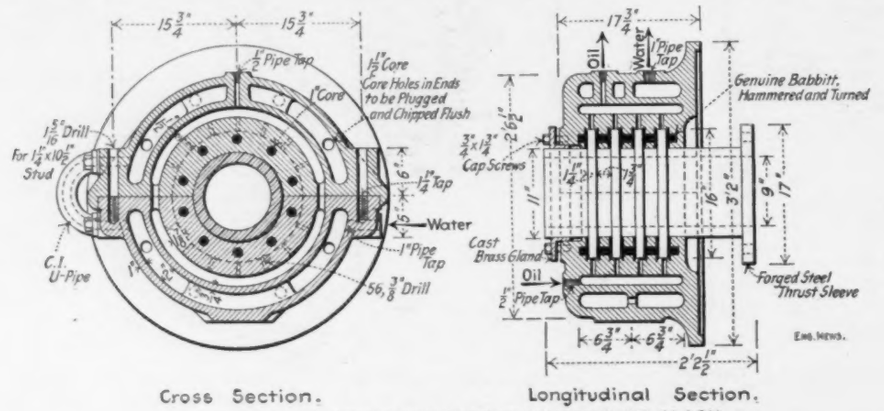


FIG. 6. DETAILS OF COLLAR-SLEEVE AND THRUST BLOCK.

The balancing piston is so proportioned and the pressure supply pipe is throttled to such a point as to give exact balance, (i. e., with zero thrust in the thrust-bearing) at about half to five-eighths the full output of the wheel. At larger power there will be an unbalanced thrust to the right, and at smaller output to the left, which are taken by the thrust-bearings. The maximum thrust on the collars is about 25,000 lbs. The collars are 2½ ins. high (2⅜ ins. effective), by 13½ ins. mean diameter, giving a total effective bearing area on four collars of 418 sq. ins. The maximum collar pressure is thus about 60 lbs. per sq. in.

The regulation of the turbine is of the swivel-gate type. The guide-vanes are pivoted, and by means of an operating ring connected to all the guide-vanes they are actuated by the governor, increasing or diminishing the width of the guide-passages according to the change of load. The general arrangement of the mechanism is made clear by the general drawings, Figs. 1 to 4, but several details deserve special attention.

The guide-vanes are integral with their pivots or spindles, being steel castings finished all over

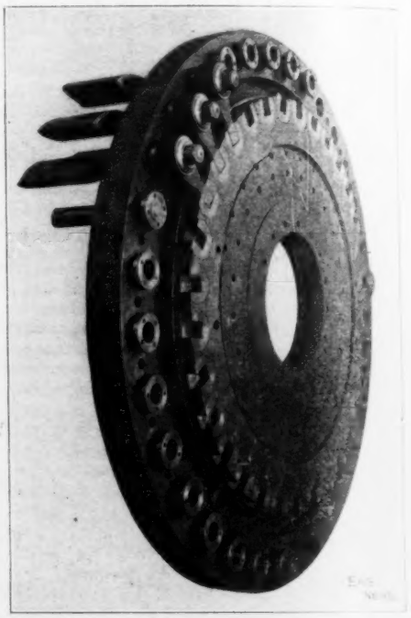


Fig. 8. View of Gate-Operating Ring with Three Guide-Vanes in Position.

by turning and planing. The spindle is carried in the housing in tight-fitting brass sleeves, as shown by a detail sketch in Fig. 5, which serve as bearings and as packings. The forward end of the spindle does not extend through the casing, and to prevent any pressure existing behind it due to the unavoidable leakage of pressure water past the sleeve, which would produce a heavy end thrust on the spindle, a drain-hole to the draft-tube is provided. The rear end of the spindle projects through the casing and carries an arm engaging with the operating ring. The packing sleeve on this portion of the spindle is supplemented by a stuffing-box back of it, and a separator in the packing of this stuffing-box provides a drain-space which is drained through a groove on the inner face of the end-plate casting to the space back of the turbine runner.

The detail of the connection between the guide-vane spindles and the operating ring is an interesting design. Each spindle has keyed to it an arm, outside of the turbine case, which is finished with a straight rectangular end. The end of the arm fits with a close sliding fit in a cylindrical block set in a cylindrical recess in the rear face of the operating ring. Fig. 7 shows this construction, which gives a connection hav-

ing ample bearing-surfaces at all point of motion, thereby ensuring minimum wear at this vital point. Fig. 8, a view of the operating ring and several of the guide-vanes, all set in place in the turbine endplate, shows the assemblage.

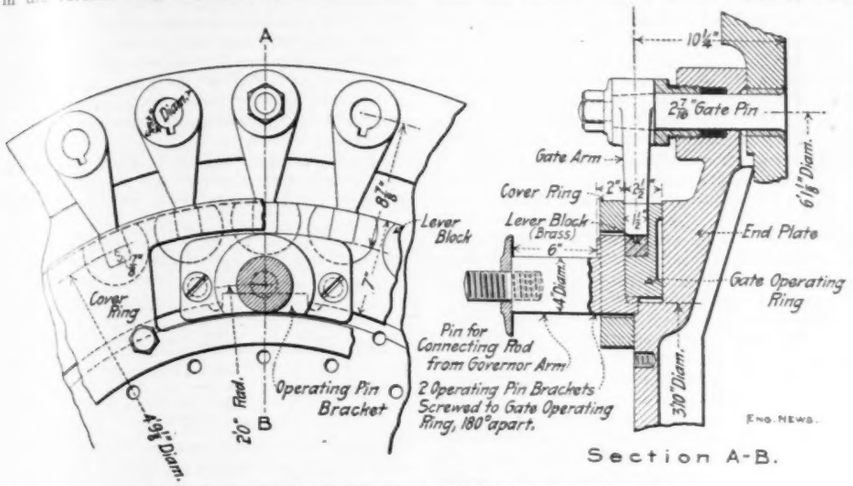


FIG. 7. DETAILS OF GATE-OPERATING MECHANISM.

An annular cover-plate over the ring, enclosing the hinge-block mechanism, is not shown in Fig. 8. Two blocks attached to the operating ring at diametrically opposite points project through slots in the cover ring and carry pins which serve as attachment for the operating rods from the governor sector. The length of the slots defines the maximum motion of the gate operating ring. The gate mechanism as described is patented by the Platt Iron Works Co.

Accurate tests of the turbine have not been made. After it was installed, a test for efficiency was made, in which the output was measured on the electrical side, while the input was measured by the drop of head across the headgate, whose opening was accurately known. This test showed an efficiency, at 10,000 HP., of 84%; this figure is subject, of course, to the inaccuracy of the method used for measuring the water consumption.

A test for maximum power was made by opening the swivel gates to their maximum opening, the governor being disconnected, as under this condition the gates could be opened slightly wider than the design called for. The test had in part the object of subjecting the machine to maximum strain. The highest capacity, registered by calibrated wattmeters, was 8,250 KW., or 11,000 HP.

As soon as this part of the test was finished, Mr. Harrisburger, the General Superintendent of the Seattle & Tacoma Power Co., was desirous to know how the power unit, both turbine and generator, would behave when let loose and run empty with wheel gates wide open and no load whatever on the generator. Previously being assured by the manufacturers of the turbine as well as of the generator that there was no objection to such test it was decided to try it. The wheel was loaded up to its full capacity, the governor was disconnected and then the switches were pulled, leaving the wheel as well as the generator without load and without control. The highest speed thus obtained was registered by the tachometer as 505 r. p. m. Mr. Harrisburger, to demonstrate his faith in the machinery, was standing close to the water wheel while it was running at this speed.

The turbine described was, as already noted, built by the Platt Iron Works Co., and was designed by the writer, then Chief Engineer of that company. The machine weighs about 190,000 lbs., excluding the steel supply pipe and draft-tube.

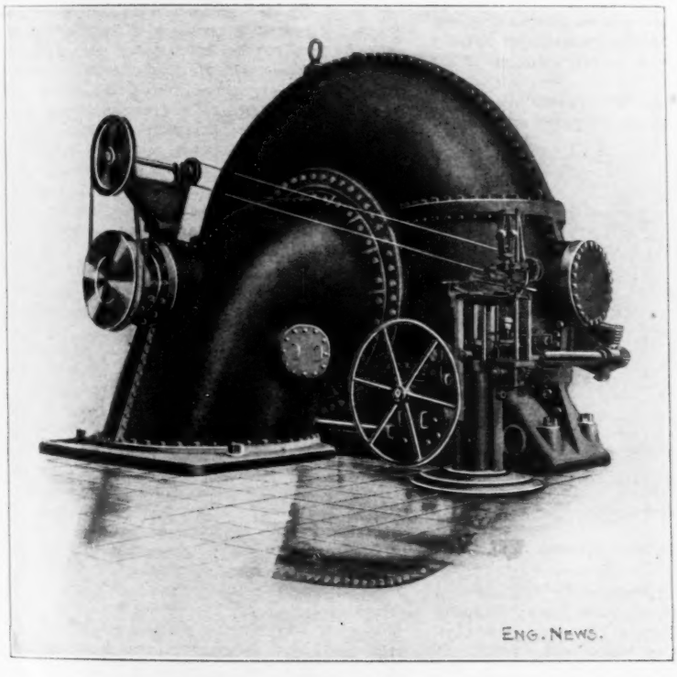


Fig. 9. Front View.

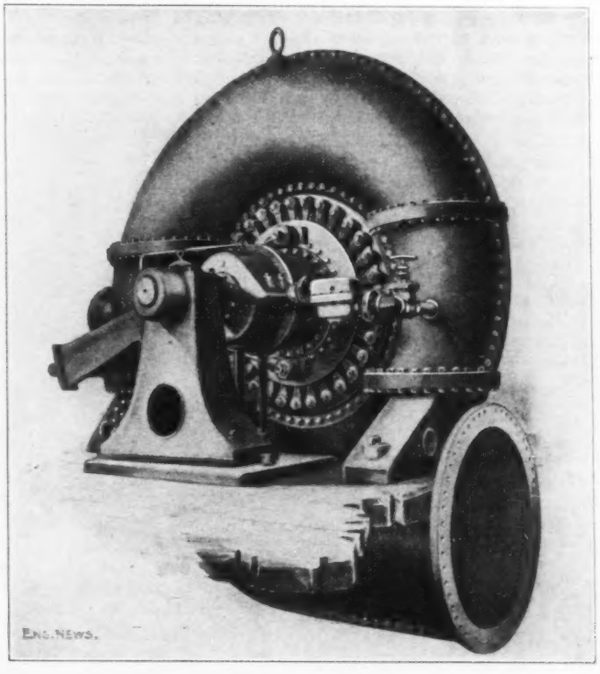


Fig. 10. Rear View.

FIGS. 9 AND 10. TWO VIEWS OF 10,000-HP. TURBINE AT SNOQUALMIE FALLS.

SOURCES OF THE PUBLIC WATER SUPPLIES OF WISCONSIN.*

There are 270 cities and villages in Wisconsin, of which 120 have been provided with public water supplies. Of this number 50 are supplied with artesian water, 30 with ground water,† 14 with lake water, 12 with river water and for the remainder the supply is not definitely known. Of the cities using river water only 6 use it for both fire and domestic purposes, the others, for fire protection or mechanical purposes only.

River water is purified by filtration at Appleton, Merrill and Stevens Point; all the others have only strainers or use raw water. It is possible to obtain much better results by filtration than are obtained at the above named places, and it is possible to make a fairly good potable water of the river waters of the state, although the expense of the undertaking might not justify the attempt where other sources are available. Nearly every stream used for any kind of a supply is contaminated to some extent with sewage and it is very evident that they will be more so in the future.

Lake water is treated at Superior and Ashland by sand filtration, and at Oshkosh and Marinette by mechanical filtration. All other lake cities use raw water. Nearly all lakes used as a source of supply are also used for the disposal of sewage. But in the case of Lake Michigan the pollution does not reach the intakes of the supplies except during storms.

Of 55 artesian well supplies, 27 are from the Potsdam sandstone alone, 10 from the St. Peters alone, while 16 are from both. It has been shown from the data collected that the Potsdam and St. Peters sandstones are very uniform in texture and porosity, and transmit almost a uniform quantity of water under similar conditions.

Of all the classes of sources herein described, the ground water supplies appear to have a greater percentage of cities short of supply than any of the other classes. They are also poorer in quality than any of the sources in the general division of ground waters, but not as poor as some of the lake and river waters. The most

*Summary of a monograph on "The Sources of Water Supply in Wisconsin," by William Gray Kirchoffer, C. E., published as Bulletin No. 106 (Engineering Series, Vol. 3, No. 2, pp. 163-267), University of Wisconsin.

†By the term, ground water, is meant the water which percolates through the surface material overlying the rock, such as soil, drift and alluvium, and through the surface layers of the rock where they are close to the ground surface. These waters are usually under little or no static pressure and do not rise materially above the general level of the ground water in an opening made in the ground water horizon. (P. 190 of monograph.)

serious trouble with artesian water in respect to quality is hardness and the presence of iron.

In regard to the amount of water used from the various sources; lake waters rank first, artesian waters second, ground waters third, and river waters fourth. As to a sufficiency or quantity lake waters rank first, river waters second, artesian waters third, and ground waters fourth; and as to purity artesian waters rank first, lake waters second, ground waters third, and river waters fourth.

SHOP HINTS FOR STRUCTURAL DRAFTSMEN.*

By John C. Moses,† M. Am. Soc. C. E.

II.—The Draftsman and the Bridge-Shop.

In a previous article attention was directed to templet making, and some of the ways were pointed out in which the draftsman could decrease the expense of that part of the process of manufacturing structural work. After the templets are made the steel must be cut, punched, assembled and riveted. In this work, also, it is true that the draftsman can often reduce costs by understanding the methods of the shop and planning the work with these methods in view. Two ways that are equally good from a theoretical point of view may differ widely in point of economy; even where one way is theoretically or aesthetically better than another, the advantage may be gained at a cost that is much greater than the designer would consider himself justified in paying if he knew of it. And sometimes carrying out a theoretically better plan necessitates a kind of shop work that is not as reliable as are the ordinary methods, and the final result is a poorer instead of a better product. The following notes on the principal operations in the shop will show some of the considerations that should govern the draftsman in his work.

SHEARING ANGLES.—Angles are cut to length and to the required bevels at the ends on a special machine having a knife with two cutting edges, one horizontal and one vertical, as shown in diagram by Fig. 1. This knife can cut both legs of an angle at the same time. The whole machine stands on a turntable, and the angles to be cut rest on horizontal skids. Square cuts are made by placing the machine at right angles to the stock on the skids. In this case the templet need only indicate the length, and the marker draws a square line on the angle with his try-square. A permanent guide line on the machine shows when it is set square to the work. To make bevel cuts, the machine is swung to the bevel given by the templet, or the angles themselves are swung around when not too long or heavy. If a "front cut" (see Fig. 2), the angle is supported by the table of the machine, as in square cuts, but if a "back cut" is wanted the angle must be put into the other side of the knives, on account of the housing, and is generally carried around to the back side instead of turning the machine entirely around. In either case the degree of the bevel is limited by the construction of the machine, as an examination of Fig. 1 will show. For front cuts this bevel varies with the width of the leg cut, and a record of these limits should be in the drafting office. The limiting bevel for back cuts is the same for all sizes.

The bevel cut is made on the horizontal leg of the angle and the vertical leg is left square. If the bevel is a front cut, the angle can be turned over and the other leg also be given a front cut, but one leg cannot have a back cut and the other a front cut, nor can both legs have back cuts.

If bevels outside the limits are necessary, the angle is cut square and then the beveled leg is cut on the plate shear, or else is punched off. This leaves the square leg with a square face. Bevels inside the limit, on the other hand, should be shown as cutting the square leg to a sloping face to save a second cut on the machine.

Fig. 2 shows two ways of drawing an angle for a lateral brace or truss member; the lower sketch shows the angle with square ends, the upper one the same angle with bevel ends. Where the ends are beveled the operations in cutting the angle in the shop are as follows: A cutting templet must be made for each end in addition to the regular pole, thus doubling the templet work. The angle is first cut square to length, then carried around to the rear of the machine and the back cut made; then to the front of the machine and one front cut made; then turned end for end and the other beveled cut made. If the angle is long or heavy, the machine must be swung for each of these different cuts. Care must also be taken to cut the bevels the correct hand. The economy of cutting angles square will be evident, even if the gusset has to be made larger. The effect on the appearance of the work is much more apparent on the drawing than on the work itself, since the angle leg is flat against a large plate of the same color after painting, and is so much less conspicuous than the outstanding leg that it will not be noticed. In this case, as in many others, the draftsman should frequently examine finished work in order to have a true conception of the result of his drawing.

SHEARING PLATES.—Small gussets are generally cut from large pieces on a shear arranged as shown by Fig. 3. Frequently a number of them can be made with two sides parallel, of the same width; they can then be cut from long plates of that width with few cuts and little waste. When too irregular in shape for this, they should be planned with as few sides as possible, as every side represents a cut of the shear. The size of plate from which they are to be cut should be kept in mind and the gussets planned to use up the material with as little waste as possible. Corners much less than 90° should be avoided, as the plate will curl when sheared and not fit snugly to the adjoining parts.

Large gussets may be ordered from the mills cut to the shape desired. They are then known as sketch plates, and an extra charge of one-tenth cent a pound is made for them. It is a good plan to figure their cost and compare it with the cost of a rectangular plate of sufficient size, crediting to the latter cost one-half the value of the plate cut off as good material for future work, and the other half at scrap value. The rectangular plate will often be found the cheaper. Plates ordered "sketch" from the mills generally have to be trimmed again on the shears in any case, and are quite often cut wrong or delayed in shipment.

An examination of Fig. 3 will show that long plates to be cut in two on a bevel are limited in position by the frame of the shear. The wider the plate the less the possible deviation from a square cut. This must be kept in mind in ordering stock, and a diagram of the shop shear should be among the draftsman's data.

Reëntrant angles on gussets should be avoided, as they cannot be cut by the shear, but must be punched out. The shear will not cut off a strip of less width than about half the thickness of the metal, and there is, of course, a limit to the thickness it is safe for any given machine to cut.

CUTTING BEAM WORK.—Beams, channels, zees and tees are generally cut in pieces by sawing. This is a much slower and more expensive process than shearing plates and angles, and for that reason it is best to order them to the lengths wanted. They can be trimmed to exact length by the coping machine, which has a heavy square punch for removing portions of flanges, and also shearing knives for cutting the web after the flanges are removed. Beveled cuts on beams and channels are expensive and troublesome. The smaller sizes of beams and channels can sometimes be beveled in the plane of their webs by first coping off one flange and then cutting the web and other flange on the angle shear. This, of course, means carrying the stock from one machine to the other and handling it twice. Channels that are too large or too sharply beveled for the limits of the angle shear, and beams of all sizes, can have one flange removed by the coper and then have the web sheared to a bevel. This leaves them as at A in Fig. 4, with one flange cut square. This blunt corner will rarely be objectionable, and the draftsman should draw them that way when possible, instead of like sketch B. Beams or channels beveled transversely to their webs are especially objectionable to the shop. Instead of cutting the flanges on a bevel, they can be coped as shown by the sketch C in Fig. 4. When the engineer insists on a sloping cut, as at D, Fig. 4, the shop is driven to heating the end of the beam in a furnace in order to make the cut by hand at a reasonable cost. In most cases this is certainly more objectionable than the sacrifice in appearance where the beam is coped like the illustration. When coping to obtain clearance in this way it is not necessary to cut off the flange absolutely flush with the web. It is well to say "Cut flush" when this is necessary, and have it understood that an ordinary cut will do in other cases.

LATTICE BARS.—Lattice bars are made by a punch that forms the rounded end and punches the hole on one end of each of two bars at one stroke of the machine, as shown by Fig. 5. A gage is set to give the length and no marking of the iron is necessary. The draftsman should therefore have as few kinds as possible and should avoid any special shape of ends.

FLANGE PLATES.—Girder flange plates are frequently drawn with the corners cut at 45°. Where double gage lines are used in the angles and the rivet pitch is small, it is necessary to put

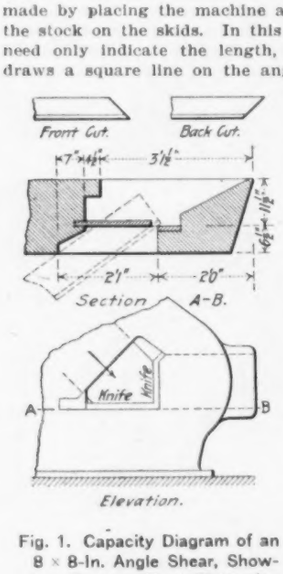


Fig. 1. Capacity Diagram of an 8 × 8-In. Angle Shear, Showing Factors That Determine Limiting Bevel Cuts.

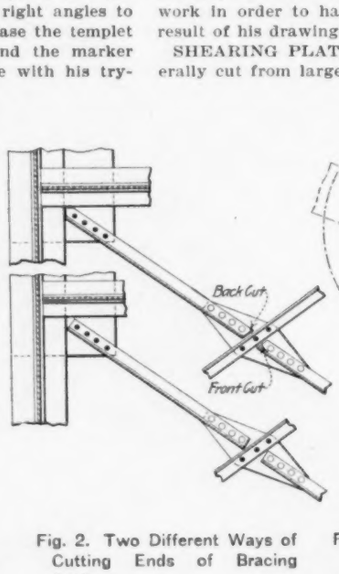


Fig. 2. Two Different Ways of Cutting Ends of Bracing Angles.

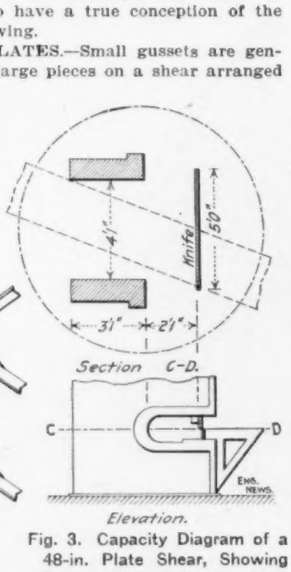


Fig. 3. Capacity Diagram of a 48-in. Plate Shear, Showing Factors That Determine Limiting Bevel Cuts.

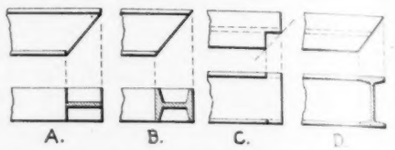


Fig. 4. Different Ways of Beveling Ends of I-Beams and Channels.

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†Engineer of Construction, The Boston Bridge Works, Inc., Boston, Mass.

the end rivets of the flange plates on the inner gage lines and cut the corners to clear the rivets on the outer gage lines. But where the pitch is sufficient a better plan is to have the rivets on the outer gage lines and cut the plates square. This not only saves a large amount of shop labor, but makes it much easier for the track man to frame his ties in the case of a deck railroad girder. As far as looks are concerned, the remarks on shearing angles square instead of beveled are applicable.

STRAIGHTENING.—Long plates have to be straightened edgewise after delivery at the shops. If used for webs of riveted members they should

Fig. 5. Method of Cutting and Punching Lattice Bars.

be narrower than the outside dimensions by ¼ to ½ in. to allow for slight crookedness. Much work on the straightening press will then be saved, as well as probable chipping on the member before the flange plates and gussets can be put on. There will also be less danger of damaging the steel by trying to get out small kinks. Web plates of girders having full length top plates to protect them from water should be ordered *one-half inch* narrow, or they will have to be sheared or chipped in places.

PUNCHING.—When more than one size of holes is called for on a piece, that piece must be handled once for each size, and the cost of punching each time will be practically as great as would be the cost of punching all the holes the same size. This is very nearly true, even if only one hole of a different size is called for. Further, it costs as much to drive a five-eighths rivet as a seven-eighths rivet, and generally more of the smaller size will be required than of the larger. When driving the rivets a change in size means stopping the work to change dies, or else going over the work a second time. Thus, both punching and riveting considerations speak for uniformity in size of holes.

It will generally be cheapest to widen lattice bars that are so narrow as to require smaller rivets than the rest of the work. If some of the holes must be reamed for bolts, make the bolts ⅛-in. larger than the rivets. If more than one size of holes must be used in the job, make the change in the small pieces, as they can be handled easier from one punch to another. Holes for spiking piece bolts and tie rods that are smaller than the rivets used in the connections can be the same size as the rest of the punching, if a washer is used under the nuts. It will often pay to increase the width of leg of some pieces that are designed with regard only to the stresses, in order to enable the regular size of rivets to be used. Holes larger than the usual maximum allowable in a given size of beam or channel may be put into beam flanges at the ends of the pieces, where section is not required for strength. Slots in webs should be made the same width as the diameter of the punch.

Uniform gages for all holes in the leg of an angle are very desirable. When but one gage line is used the templet consists of a pole that is laid beside the angles, and chalk lines are squared across from the marks on the pole. A gage is set on the punch, and the holes are punched without center punch marks being made. When flat templets are used and the holes marked with a center punch, the use of gages is still advantageous, aiding the men at the punch to do more rapid and accurate work. A hole not on the regular gage, however, must be specially marked and extra care taken to avoid punching it like the regular holes.

Punches are likely to break when of smaller diameter than the thickness of metal punched. Holes should not be put on a joint so as to make half holes in the abutting pieces, as the tit on the punch will force the piece to one side and make a bad hole, besides soon breaking the punch.

ASSEMBLING.—In some shops the men that mark the material for cutting and punching are supplied with the drawings. The templets in this case do not give size of holes, countersinking, allowance for facing, etc., and the markers have to spend considerable of their time studying the drawings, and are high-priced men. Other shops do this work in the templet shop and put all information needed by the marker on the templet. The drawings are then not used in the shop until the punched material is assembled for the riveters.

The cost of assembling, or "fitting up," varies greatly with the character or diversity of the work and may be as great as the cost of punching. The men engaged in this work want clear drawings, with the various plans and elevations shown in proper relation to each other. End views rather than sections should be given, for a man can take his drawing around to the end of a piece and compare it with the work, but he cannot see a section without the use of a good deal of imagination. Top, bottom and end views should always be placed as shown in Fig. 6, and not (as is still taught in some schools), with the top view below and the bottom view above. No shop man wants the right-hand end view of a piece at the left of its elevation, and no one else will that has ever tried to use such a drawing in the shop. The one exception to this rule is that the bottom flange of a girder is generally shown by a sectional plan, and this is done more as a concession to the draftsman than by wish of the shop.

Columns are assembled and riveted in a horizontal position and should be drawn in that way, with the bottom to the left. If drawn vertically every man in the templet department or shop will have to turn the drawing around when he uses it.

Each drawing should have on it a list of all the complete members shown on the sheet. This list should give the number wanted, name, hand and shipping mark in this way:

Two End Posts wanted, as shown, Marked LoU2R.

Two End Posts wanted, opposite hand, Marked LoU2L.

The foreman checks off against this list as he fits up the work. The list should have a descriptive name for each piece as well as a shipping mark, and this name should also appear on the shop bills and shipping invoices. All notes should be written in language that is not ambiguous through being too much abbreviated; notes are often put on in such poor English that they can be read in more than one way.

Open holes, flattened and countersunk rivets should be shown in such a way that they can be marked for the riveters without having to be located by measurements on the iron. The required clear spaces between chords, and the extreme width of entering members should be given directly, and should not have to be obtained by adding up other figures. The distance between open holes on end connections should be given similarly. These distances have to be checked on the iron before riveting is begun; wooden blocks are often bolted into the chords and clamps used on entering members to keep such pieces to the proper width while being riveted up. Note dimensions that must not vary over 1-16-in. as "exact," this being the practical limit of accuracy in structural work. Notes should be given positively, stating that a "Bracket goes on Columns 1, 2, 3, 4 and 6," and not "Omit bracket on Column 5 only." Small sketch views should be used when pieces shown on the same drawing differ much in arrangement of attached parts. It is much cheaper to make them than to have the assembling gang losing time puzzling over complicated notes that are more or less hidden by paint and grease spots, due to handling the drawings and the steel work at the same time.

Center lines, gages, bevels, etc., do not appear on the punched material, and it is often difficult to tell from inspection which way a piece should go. The spacing of the shop rivets can often be made in a way to prevent reversing a gusset or riveting an end connection angle upside down. If the two legs of a connection angle are alike except for a slight difference in gage, the gage of the shop driven leg should be changed to be either the same as the other leg, or else markedly different from it.

RIVETING.—Shop rivets are generally calculated at higher values than field-driven rivets, it being assumed that they are driven by machine riveters capable of exerting heavy pressure. It is important, therefore, to avoid placing them in positions that cannot be reached by these machines. Rivets that cannot be reached by the machine riveter are left to be driven later by hand at a much greater cost.

Fig. 7 shows the factors determining gages and clearances for such machines when used with full-sized dies. When rivets must be driven on narrow gages the corner of the die is ground away to clear the fillet of the angle. But the rivet head will very likely be forced to one side and not be concentric with the shank, and the dies will soon lose their proper shape and have to be repaired or renewed. For the same reason flange rivets should not be placed too close to stiffener angles or lateral gussets.

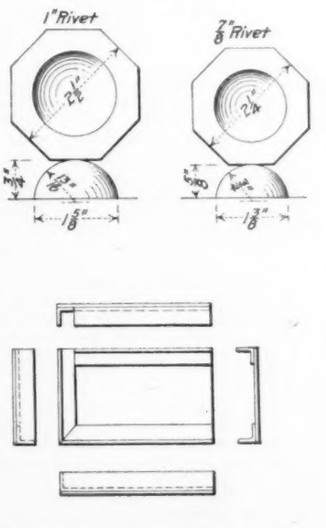

Fig. 6. Proper Arrangement of Views in the Representation of an Object

Fig. 7. Sketches Showing Factors Which Determine Minimum Gage in Angles, with Rivets Driven Opposite.

Countersunk or flattened rivets require changing dies, and if countersunk rivets have to be flush they must be chipped by hand. When they occur in shoe plates they are generally driven by hand and chipped flush while hot. As they are not required for carrying stresses in such places, their number can often be reduced to advantage.

Draftsmen have been known to place flange plate rivets directly under the outstanding leg of a stiffener, countersinking them in the angle. This practice is very objectionable, for a number of reasons. It involves countersinking the hole, thus removing more metal than was figured on by the designer, and the rivet must, of course, be chipped. The most serious objection, however, has to do with the order in which the riveting is done. It is customary in plate girder work to first bolt the flange-angles, stiffeners, fillers and splice plates to the web and drive all the rivets whose axes are at right angles to the web. Then the flange-plates are put on and their rivets driven with a different machine. But if the flange-plate rivet comes under a stiffener

age at last—which would gradually take place, coming up perhaps half a mile below the dam.

There is nothing uncommon about water flowing through a half mile or a mile of such material. And if that dam were built and water were discovered coming up through the earth on the down-stream toe and below it, it would be a source of continual apprehension as to what the result was going to be—whether the dam would not break out and discharge all its contents, as other dams have done, like this one of which I have just given the name, the Mill River dam.

SENATOR MORGAN.—Springs and rivers flow beneath mountains, do they not?

MR. BURR.—They do.

SENATOR MORGAN.—And the mountains stand, as in the case of the Humboldt River?

MR. BURR.—They do; they stand. Sometimes, however, large banks of earth are carried away by precisely those springs. I have seen such a number of times.

Of course in such cases the water comes from not great depths, but it has through long ages made a channel for itself, and it does not flow under any great head.

SENATOR MORGAN.—The cores put into earth dams, I suppose, are intended to prevent this very underflow?

MR. BURR.—They are.

SENATOR MORGAN.—Do they necessarily rest on the rock bottom?

MR. BURR.—Not necessarily. They may rest on a truly impervious bottom.

SENATOR MORGAN.—A clay bottom, for instance?

MR. BURR.—Yes; or a hardpan bottom.

SENATOR MORGAN.—Or hardpan?

MR. BURR.—Yes.

SENATOR MORGAN.—It is not necessary to carry them down to the rock in order to make them useful?

MR. BURR.—Not necessarily; no.

SENATOR TALIAFERRO.—Did not this Mill River dam have a masonry core?

MR. BURR.—That had a masonry core. Water found its way under that also, but it was not carried down to rock. It was constructed in an earth excavation.

I only mention the Mill River dam because I happened to make a note of it; but there have been other failures where there was no masonry core.

Much has been said about the weight of water on the bottom of the lake compressing it and making the material more impervious. I grant, as every engineer must (or any sensible man, it seems to me), that wherever weight is put upon a compressible material it will make it more dense. But there is one very pertinent observation to be made in connection with that pressure here—that in consequence of this irregular variation of material that settlement would not be uniform. It would be more at some places than at others, and that would cause a break in the bottom of the reservoir wherever that variation existed and form an opening for water to find its way into the deeper pervious materials.

The dams on the Pacific side are smaller and the risks, perhaps, may be of less magnitude; but they are of the same character and there is the same objection to them, in my opinion.

This dam between La Boca and the high ground opposite would be founded largely upon the most slippery kind of mud. Any one who has been there and seen the bottom of the Rio Grande estuary exposed at low tide I think will agree with me that it is a very lubricating material; and if you were to put a bank of earth on it, even if it were half a mile thick, I think it would be in grave danger of being pushed out bodily. I have seen such banks on a smaller scale pushed out in precisely that way.

I would make no new experiments on the Isthmus of Panama in connection with this work.

SENATOR MORGAN.—We have made one that has turned out to be very expensive by adopting a plan to build the canal with the key of the situation at Bohio. We have not commenced to build the canal yet, if I understand it.

MR. BURR.—I do not understand, Senator, to what you refer. The Bohio dam, you say?

SENATOR MORGAN.—Yes.

MR. BURR.—That was a perfectly safe type of dam, Senator. I should stand by that.

SENATOR MORGAN.—If you had a safe place to put it.

MR. BURR.—That is precisely what I am advocating now. I am consistent with what I did then. Mr. Morrison wanted to experiment in just that same way and I opposed it.

SENATOR MORGAN.—The plan was good enough, but you did not have any place to put it.

MR. BURR.—Well, I would not hesitate, Senator, to make a perfectly safe dam at Bohio.

SENATOR MORGAN.—Now?

MR. BURR.—Now; with the depth of 168 ft. I should not carry the masonry core all the way down, but I would make the dam safe.

SENATOR MORGAN.—How much money would you spend on it?

MR. BURR.—Not such a great amount. I would make it within our estimate.

SENATOR MORGAN.—And you recommend to-day the plan which your committee reported for a dam at Bohio for that canal?

MR. BURR.—If I were going to express my preference for a lock canal, Senator, I should say just what I said and argued for in the consulting board, that the canal should be made with the sea-level section on the Caribbean side carried to Obispo and on the Pacific side to Miraflores, and then lock up from those two points to perhaps a 60-ft. summit level, possibly an 85-ft. summit level, if you please; but if a lock canal were to be built, in my judgment that should be its plan.

SENATOR MORGAN.—Where would you put your dam to make that 65-ft. elevation?

MR. BURR.—I would put the controlling dam at Gamboa, just where the sea-level plan places it.

SENATOR MORGAN.—Why would you not put it at Bohio?

MR. BURR.—With the subsequent investigations, made since, I should prefer to carry the sea-level section to Obispo, or nearly to Obispo; perhaps not quite, but very nearly.

SENATOR MORGAN.—But Wallace's investigations have disproved the practicability of putting it at Bohio, have they not?

Mr. Burr.—Oh, no; I should not be willing to admit that, Senator. His investigations have shown that it would be more difficult to build there than was supposed.

SENATOR MORGAN.—They have shown 40 ft. greater depth?

MR. BURR.—Yes.

SENATOR MORGAN.—And that would be with caisson work?

MR. BURR.—Oh, I should not take caisson work down to that depth.

SENATOR MORGAN.—What would you do, take ice down there and freeze it, on Hanna's plan? [Laughter.]

MR. BURR.—Well, the freezing process might be applicable, but I should not expect to use that. What I should expect to do would be to go down as far as it is feasible to go with the pneumatic process, perhaps to 140 ft., with the head of water above pumped down by making a great pit around the caisson; and then, from the lowest point reached by the pneumatic process, I would put one or preferably two lines of heavy sheet piling from that point down to bed rock, 30 or 40 or 50 ft., if you please, which would make a complete curtain and shut off all possible seepage.

SENATOR MORGAN.—Then you would build your dam on top of that sheet piling?

MR. BURR.—On the top of that I would place my dam. The function of the sheet piling would not be to support anything, but simply to make a cut-off for the water; and if it were found impracticable to run that sheet piling all the way across, you could inject cement in that gravelly material overlying the bedrock, and so close up any possibility of material seepage.

SENATOR MORGAN.—And build a stone wall out of cement below there?

MR. BURR.—And force in the cement; yes.

SENATOR MORGAN.—In constructing the Gatun dam, could they not use this sheet piling as well as you could at Bohio?

MR. BURR.—It can be done. Properly done, it would add largely to the cost; I do not know how much, but it could be done. I do not know whether you could carry the sheet piling down to 258 ft. I think that is a matter of grave doubt; but you might do something——

SENATOR MORGAN.—You could do it with the assistance of the caisson process to get down to where you would start to drive your sheet piling, or to insert it through pumping out the material?

MR. BURR.—You could use sheet piling to a great extent, so far as it would go, and then resort to some such process as injecting cement—something of that sort.

SENATOR MORGAN.—After all, that would be about the best plan, would it not?

MR. BURR.—It would if you could depend upon closing up everything; but when you inject cement in gravelly material you hope to close everything, but you cannot always depend upon doing it.

SENATOR MORGAN.—If you put pressure on it, you can do it.

MR. BURR.—You can get a large amount of stuff into a very close material; but whether you could close up a geologic valley, so-called, two or three hundred feet wide—oh, you could doubtless close it up near enough for all practical purposes.

TESTIMONY OF MR. FREDERIC P. STEARNS.

MR. STEARNS.—Before visiting Panama I had considered the subject of placing a dam at Gatun, and made drawings and presented the subject to the Board of Consulting Engineers on the basis of the borings that then existed. They seemed to be sufficient to show clearly that it was feasible to build a dam at that place. Then, as it was desirable to have more borings, I prepared a draft of a cablegram which was sent to the Isthmus, asking for additional borings directly at the site of the dam.

In the engineer's office at Gatun they had the samples that had been washed up from the ground by those borings, and very generally they showed, if not in all cases, that there was a large amount of clayey material with the material that was washed up.

SENATOR TALIAFERRO.—Do the borings all go down to rock?

MR. STEARNS.—Very nearly all. I think there was one that did not go down to rock, but I am not sure but what it was repeated afterwards and carried to rock.

SENATOR MORGAN.—When you say rock do you mean real solid rock or indurated clay?

MR. STEARNS.—I mean indurated clay.

SENATOR MORGAN.—You should call it by that name, because it is not rock.

MR. STEARNS.—It is rock, to all intents and purposes, as a foundation or for stability and standing. Now, I am referring to the indurated clay that is between Colon and Gatun; and at this point it is different from some of the other indurated clay in the Culebra Cut, which dissolves in water. This does not dissolve, according to my information.

Mr. Maltby told me that the French excavated a dry dock with vertical sides, and that I think must have been back in 1880 something—certainly it was the old canal company that did it—and that that retained there its vertical sides without any trace of action by the weather upon it at all. That is not true of some of the indurated clay in the Culebra cut.

The alluvial material which fills the gorge at Gatun was to a large extent brought in after subsidence of the land, so that the currents were checked by being below sea level.

The evidence of borings, both at the Gatun dam and within a mile or two of it upstream, is all to the effect that the material overlying the rock was deposited in sluggish currents. Mr. Wallace, in a statement before us, suggested that—and I think it is generally recognized—that the material in the lower part of this estuary was formed when the gorge sank below the level of the sea, and was then filled with fine material. There is coarse gravel up at Gamboa, and just below Gamboa there is one place there where there are stones, I should say, as big as my two fists.

Coming farther down the stream where the Chagres is crossed by the railroad one can see gravel bars; and further down they are not as plain. Going still farther down, they disappear.

Mr. Nichols, who had charge of the Gatun borings, said, as a part of his report to the Isthmian Canal Commission: "In the test holes no boulders have been encountered." These are the test holes in the vicinity of Gatun. He also says: "In the valley the material above the rock is composed of clay—clay mixed with sand in proportions varying from almost pure clay to nearly pure sand, and some streaks of shells and vegetation in sundry stages of decomposition, which indicated a fine material deposited in sluggish currents."

The thirteen borings west of the island showed sand without admixture of clay for 10 ft. in one boring that was 187 ft. deep; for 1 ft. in a boring that was 146 ft. deep; for 8 ft. in a boring that was 205 ft. deep, and for 5 ft. in a boring that was 193 ft. deep. In none of the others was there sand without clay noted. For records of gravel, gravel was noted in two cases for a depth of 1 ft. in each case.

Mention has been made of the fact that wood is encountered, and also that vegetable matter is encountered in some places. I rather welcome the appearance of vegetable matter, because it was evidence, as it seemed to me, of the fineness of the material that was deposited there. Moreover, vegetable matter mixed in with sand makes it impervious to water.

If one were building a masonry dam on top of this alluvial material, then it would be a very serious matter to have organic matter in it, because it would settle; but in building a dam such as is proposed, with a great mass of earth, the weight on it is such that if that material settles, and it will if there is some vegetable matter there settle a little, the tendency would be rather to crowd that vegetable organic matter in with the grains of sand, in between the grains of sand, and make it all the more tight.

In some of this material, not at Gatun, but on my own work, that had organic matter in it I put some jackscrews on it and brought a pressure of 20 tons on a small area—not as much per square foot as this would be—and when I compressed that material having the organic matter in it it made it so solid that, while it was soft surface soil at first, before pressure, after it had been pressed with that weight it was so compact that one could hardly drive a pick into it. With this enormous weight, if there is any compressible material, it will tend to crowd it together and make it solid.

SENATOR DRYDEN.—You spoke of the sinking in this dam. Whatever it might be it would occur during the process of construction and by the time of completion?

MR. STEARNS.—Certainly.

SENATOR DRYDEN.—Would there be any danger of its going on after the dam was completed?

MR. STEARNS.—I think it might continue somewhat after. I once built a bank that was 30 ft. high on a great depth of mud. The mud was composed of organic matter and clay that had been washed from neighboring hills

leg, that stiffener will have to be left off until the flange rivets are driven. Then the girder will have to be taken back to the first machine for driving the stiffener rivets, or they will have to be driven by hand. For the same reason girder stiffeners should not be fitted down on to lateral gussets, but the gussets should be notched around the stiffener.

These examples will serve to illustrate the need of keeping in mind the order of operations in assembling and riveting. When it can be done, members are assembled complete before riveting is begun. Box chords will have their sides driven up first and then the cover plates and lattice added. When pin plates are placed over the web legs of the angles they may interfere with driving the cover plate rivets. Brackets on small columns frequently involve very troublesome riveting, and the draftsman must consider how the shop is going to accomplish what he calls for. Skew connections must often be laid out full size and a rivet of the required length be tried to see if it can be entered into the hole. Rivets in one leg of an angle may have to be countersunk or flattened in order to get rivets into the holes through the other leg. If there is room enough to get a rivet into a hole there is sufficient room so that it can be held on that end, but there must be room on the other end for driving it.

In building work it is frequently possible to so arrange connections that many of the pieces will have no riveting on them at all. For instance, a beam resting on a column bracket will have a lug on top connecting it to the column. If the lug is attached to the column the beam can go right from the punches to the shipping yard. Knees connecting purlins to trusses can be riveted to the trusses, and the same can be done with the bracing gussets.

BLACKSMITH WORK.—Blacksmith work is very expensive in comparison with the other features of structural work and should consequently be avoided as much as possible. The cost will often depend very much on the amount of duplication. When there are enough bent plates of one kind a special cast-iron die block and follower can be made, and the actual bending done very rapidly, the cost of the die being small in comparison with the saving of labor. When only a few pieces of a kind are wanted the cost per piece may be ten times as great.

Curved end angles for plate girders should be made to fit standard formers and should always be made as separate pieces and spliced to the main flange angles a foot or so beyond the bend. Allow a foot or two of extra stock for each curved piece, as the blacksmith cannot tell just where the curve will come and must trim off the ends after bending. Do not have a bend at each end of a piece, as it is very difficult to get the two bends the right distance apart. The blacksmith will have to work over it so long that the quality of the material may be injured.

Do not call for a sharp bend on a plate if a radius of ½-in. or ¾-in. can be allowed. Besides the additional labor involved, the metal on the outside of the bend will be drawn away and the plate weakened.

Top chords for light roof trusses of small pitch are sometimes made with single lengths of angles bent at the apex. Anything of this kind is a nuisance to handle at the punches, and splicing is generally cheaper in spite of the extra material and rivets required. If such bends are made they should, if possible, be to a radius that will allow them to be bent cold in the press.

CONCERNING THE GATUN DAM.

The Senate Committee on Interoceanic Canals, which is engaged in the investigation of the Panama Canal work, has recently had before it several members of the Board of Consulting Engineers, who have been examined particularly with reference to the relative advantages of the lock plan and the sea-level plan. The most notable advocate of the sea-level plan has been Prof. Wm. H. Burr, who, as our readers will recall, was committed to the sea-level plan before his appointment on the Board of Consulting Engineers. Professor Burr, in his testimony, went to far greater lengths than any other engineer in his criticisms of the lock canal project; and one feature in particular to which he raised objection was the design of the Gatun dam. These criticisms and others were effectively answered by Mr. Frederic P. Stearns when he gave his testimony before the Senate Committee at a later date.

As the design of this dam is a matter of interest to the profession generally, we have deemed it worth while to print here the testimony of both these engineers on this matter. Comment on the testimony will be found on our editorial pages:

PROF. BURR'S TESTIMONY.

MR. BURR.—The project of a dam at Gatun is a very old one. In fact, it was first broached at the International Scientific Congress held at Paris in 1879, and has been broached at various times since, but never seriously considered as a desirable part of a canal project until the considerations of the majority brought it into its present shape. As you are aware, the deepest bed rock found on the line of the dam is 258 ft. below sea level. The subsurface material penetrated by these borings is shown on the profile to disclose the variety of materials which we find all up and down the Chagres River as overlying the bed rock. There appears to be almost confusion in the way in which those finer portions of material, such as sand and clay and gravel, have been deposited along the geological valley of that river.

SENATOR KITTREDGE.—In making these borings, Mr. Burr, did any water appear at the surface?

MR. BURR.—It did, sir. In a number of those borings the water appeared and flowed slowly over the top of the pipe—that is, with a head of a quarter of an inch or half an inch or three-quarters, as it might be, at various depths, from 40 ft. down to 258 ft., showing that permeable material is likely to be found at practically any depth below the surface, and that the river water was in communication with the material penetrated by the pipes. If it had been artesian water—that is, water which had flowed from some distant and higher ground, as in the case of artesian wells—it would have come out of the pipe with a much higher head, as it ordinarily does with such wells; but it simply flowed over the top of the pipe, not far from the elevation of the water in the river. That same result has been seen time and time again in connection with our borings at the Bohio site and above. Some of the borings would not show any water flowing out of the top of the pipe and other borings would show them. There is nothing new about that. It is a rather ordinary feature of our usual experience in boring at that site; but it shows conclusively that the material penetrated by the pipe is permeable, and that water from the river enters it.

There is, however, this statement in the report of the minority, and I think it must be due to an oversight: "In the upper 200 ft. some of the later borings show fine sand, while other borings near by show clay at the same depths, indicating, as do previous borings, that the upper 200 ft. is practically impervious material."

The results of these borings which show water coming up through this material from depths of 40 ft. all the way down to 258 ft. are directly at variance with this statement. Of course water could not appear if the material were impermeable. The fact that water does flow through it and into the pipes and out of the tops of these pipes is, of course, conclusive evidence that it is permeable. I think that that must be due to an oversight.

Further, the statement is made: "There was an overflow from several of the borings which penetrated the gravelly material in the bottom of the deep gorge, although the tops of the casings were above the surface of the river."

I do not know exactly what that sentence means, but it seems to indicate that the water only flowed from the gravelly material in the bottom of the deep gorge. If that was the intention of the statement, it is without foundation, because, as you will see when you get this profile, the water flows through material anywhere from 40 ft. down to 258 ft., and doubtless if other borings were made in sufficient number water would be found at an elevation above 40 ft. below the bottom, as it has been at other places.

I think that is a very important point in discussing the safety of this proposed great dam. The results of these borings show that that material is permeable, just as similar borings show that the material is also permeable at the Bohio site. There is this difference, however, that the permeable material at the Bohio site and above the Bohio site is coarser.

The very fact that this material filling the geologic valley is of an irregular character—some impervious clay, some mixture of sand and clay, and others of open sand, such as that penetrated by these borings—is, to my mind at least, a clear demonstrative evidence that water may find its way through that material under a dam of that character, with a head of 85 ft. above it, which is equivalent to a pressure of not far from 40 lbs. per sq.

It is proposed to build this dam by simply cleaning off the surface material and then spreading the earth suitably selected from the canal excavation in layers and so building it up to a height of 135 ft., making a base something like a half a mile wide. In my judgment that is a dangerous experiment upon a colossal scale which this government is not justified in undertaking. But I wish to say at this point that I have no objection to an earth dam anywhere under suitable conditions. In fact, I think it is a most valuable type of structure, and I do not see why an earth dam may not be built of any height if properly designed and founded. But in all cases it is my clear judgment—and that based upon years of experience in connection with the underground flow of water—that suitable means should be taken to absolutely prevent anything like flow through the permeable material under the dam; and no such means are provided in this design.

The means of accomplishing that end may take a variety of shapes. It may be a masonry core wall if the bed rock is not too deep; and that is a very common type of construction in this country. There are many such dams. Again, abroad, that core wall, instead of being of masonry, is sometimes of puddled clay; but that is equally effective to prevent any subsurface flow of water. In this country we have, within the past few years, constructed dams in some localities without either a masonry core wall or a clay core wall under conditions which justified that kind of construction; and those dams have stood satisfactorily. On the other hand, we have a record of numerous failures of earth dams from water getting through them and under them, sometimes through them along a pipe laid in the embankment, and sometimes by finding its way under the dams in precisely the manner that I should apprehend might take place in this case. There are numerous instances of that kind.

SENATOR KITTREDGE.—Can you give us one or two?

MR. BURR.—I should have made a record of some of those dams, and I think I have a note of them.

A very complete account of some of those cases is found in a paper by Mr. William R. Hill, in the proceedings of the American Water-Works Association for 1902. One instance is that of the Mill River reservoir dam, at Williamsburg, Mass., which gave way on May 16, 1874. It was an earth dam, with a masonry core wall 43 ft. high—about one-half the depth of water which would exist in this dam. Its area was 114 acres. One hundred and forty lives were lost, and about a million dollars' worth of property was destroyed.

That was, of course, a much lighter dam than this, and I would willingly accord all the additional presumption of safety to this dam which goes with its greater mass. But the failure of that dam shows what may take place in this dam or any other one of a similar character under similar conditions.

Then, again, there is a reservoir which has been completed for the Borough of Brooklyn, called the Milburn reservoir, which I have inspected in the course of my professional work for the city of New York, which was completed in 1893. That reservoir was completed under engineering specifications, with a puddled bottom, designed and put in place for the express purpose of preventing leakage. Yet when it was filled with 43,500,000 gallons of water it all leaked out in ten days, and that reservoir has never been in use since. That shows what water will do when it has a chance, even in so-called impermeable material.

As stated in the majority report, nothing is more common in the experience of water-works engineers than to observe the underground flow of water through permeable or semipermeable material. In fact, many water supplies depend upon that flow, such as the Borough of Brooklyn and many other cities that take their water from the deep underground sands or mixtures of sand and earth. You will find frequently small passages of water, some of them threadlike in magnitude, almost capillary passages, and from that up to passages that flow many gallons a minute. There are wells fed by such water on Long Island; and in our recent examinations for a tunnel under the Hudson River near New Hamburg we found such a passage by diamond-drill borings, about 200 ft., if I recall the depth rightly, below the surface.

SENATOR KITTREDGE.—When was that, Mr. Burr?

MR. BURR.—Within three months—two months and a half ago; and those are not uncommon experiences. They are within almost, you may say, the daily range of hydraulic engineers or civil engineers engaged in that class of work.

You may make dams of sand, even, that will gradually silt up and be watertight. There is no question about that. You may put a dam at this place of this character, and it might stand. I do not say that it possibly would not. But I do say that as to the point whether it will stand or not it is mere conjecture. As an engineering guess, you might say that it probably would stand; but there is the very imminent engineering possibility that the subsurface flow caused by this great head of water, 85 ft., would force a seepage—a mere seep-

that there was any engineer responsible for the design or construction of the dam.

The dam was an earth embankment with a core wall 16 ft. wide on top, with side slopes of 1½ horizontal to 1 vertical. The Gatun dam is 25 horizontal to 1 vertical. The masonry core wall was 2 ft. in thickness at the top and 5 ft. 9 ins. in thickness at the bottom. It was built in dry stone and grouted, a way that engineers would not build it now. At the site of the dam there was said to be by this committee a washed porous gravel, and below that hardpan. The specifications required that the core wall should go 3 ft. into the ground, so that it would not settle, but said nothing about carrying it down to the hardpan, which was only a few feet lower, so that it would cut off the water.

The builders did not seem to appreciate the purpose of a core wall; and it was not carried down to the hardpan, but rested on this porous gravel. The slopes of the dam were found to be altogether too steep for safety, being at about the slope that gravel when dumped naturally takes itself, so that the earth had not much more than the stability required to prevent it from sliding off on the downstream side itself; and the water pressure and the water leaking through this dam, together with the tendency of the earth to take a flatter slope, caused it to slough and go away.

SENATOR MORGAN.—What was the depth of the water?

MR. STEARNS.—39 ft. It was 4 ft. below the top of the dam and had been somewhat higher. It seems to me if there is any lesson taught by that dam, it is that an entirely inadequate dam, built without engineering supervision and without engineering designs, stood for nine years before it failed. I had prepared this morning a section of the Mill River dam. (Mr. Stearns thereupon proceeded to compare this section with the section of the Gatun dam which was exhibited before the committee.)

This section has the true proportions, but it has been enlarged in the proportion of 85 to 39 to show what it would be if built to these same proportions, but with the water against it that is to be against the Gatun dam. That is the relative size of this dam, which is brought here as an instance, out of many, as to a dam that has failed, which might serve as a precedent for the failure of this dam.

I have said that in the north dike which I have built, the dam which I have built which is so similar to this Gatun dam, the water has not risen to the full height against it so as to give the 65-ft. pressure that is expected. It has, however, risen one-third of this height, and, according to the laws of filtration, there should be one-third of the water coming through there. As a matter of fact there is no water coming through there that has ever been noticed. That is, it has been tested to one-third of the pressure. Second, last year, when the water rose on it to a height of 20 ft. above the lower point, I had previously sunk in the dike an iron pipe and had measurements taken there frequently.

On the 31st of March the water was rising, and it was at this point against the dam (indicating on map), 7 ft. higher than it was in the pipe inside. That is, the water did not rise in here as fast as it rose outside. Two weeks later it had risen 7 ft. more, coming up on the dam to about this point (indicating on map), and in here, in the two weeks, it rose only 1 ft. Still two weeks later, making it a month from the first observation, the water was up at a point corresponding to this point that I am indicating on the dam, about one-third of the height, and the water in the pipe, although it was at that time, say, 12 ft. lower than the water in the reservoir, rose only 4 ft. in these two weeks.

That is, in the course of a whole month the fall from this water in the reservoir to this point in the dam [indicating] was as an average about 10 ft., showing that not enough water would filter through moderately fine sand to fill up the interstices in that sand 200 ft. away from the face of the dam.

At a point three-fourths of the way through from the upstream side of the dike the water stands in that dike higher than either on the upstream or the downstream side, showing that it is the result of the rainfall, and that the material, which is earth, is so nearly impervious that even the rainfall will not pass through it. Those observations were continued for three years, and it was shown that the fluctuations of the water in the ground in this portion of the dike corresponded to the fluctuations caused by the rainfall, and that they were caused by the rainfall.

AFTER RECESS.

THE CHAIRMAN.—Mr. Stearns, I think if you are ready to proceed now and make further statements to us, we are ready to have you do so.

MR. STEARNS.—Professor Burr, in his testimony, after citing the case of the Mill-River dam disaster, which I referred to this morning, makes this statement:

"Then, again, there is a reservoir which has been completed for the borough of Brooklyn, called the Milburn reservoir, which I have inspected in the course of my professional work for the city of New York, which was completed in 1883. That reservoir was completed under engineering specifications, with a puddled bottom, designed and put in place for the express purpose of preventing leakage. Yet when it was filled with 43,500,000 gallons of water it all leaked out in ten days, and that reservoir has never been in use since. That shows what water will do when it has a chance, even in so-called impermeable material."

Now, what is the condition, and how does that have any bearing upon the safety of the Gatun dam?

The facts are these, and they are taken from a public document entitled "The Brooklyn Water Supply," dated 1896.

The reservoir in question is a rectangular, artificial reservoir, having an area at high-water line of 48 acres. The ground on which it was built was the very porous sand and gravel found on the southerly side of Long Island. In order to make the reservoir watertight, its bottom was to be covered with 2 ft. in thickness of puddle, which it was provided in the contract should consist of 20% of pure clay, to be mixed with materials found on the ground. It is stated elsewhere that in the actual construction more clay was used than was originally provided for.

On testing the reservoir it was found to be leaky, and an investigation was made in 1894 by Mr. J. J. R. Croes, who has since that time been president of the American Society of Civil Engineers. He made tests which, he says, "appear to establish beyond question that the whole bottom of the reservoir is a filter bed, pretty uniform in character, and passing water at the rate of two-tenths foot per day under a head of 3 ft., four-tenths foot per day under 6 ft., and six-tenths under 9 ft.

It will be seen from this statement that the conditions at the Milburn reservoir were entirely dissimilar from those at the Gatun dam. The material to resist filtration at the Milburn reservoir was 2 ft. in thickness, while at the Gatun dam it is half a mile in thickness. If the comparison shows anything, it is that material which was not wholly impervious under a pressure of 9 ft. of water permitted water to pass only at the slow rate of six-tenths of a foot a day.

I have said that the speed of filtration is governed by the relation of the pressure of water to the thickness of the filtering material. Here was a head of water that was 4½ times the thickness of the filtering material, instead of being 1/25th of the thickness of the filtering material. The relations are so widely different that it has no bearing, and in any case the experiments show that the filtration was proportional to the head, which is the law I have tried to lay down to you, as proved by experiment.

The next instance which he cites to show that the Gatun dam may not be safe is that—

"Nothing is more common in the experience of waterworks engineers than to observe the underground flow of water through permeable or semi-permeable material. In fact, many water supplies depend upon that flow, such as the borough of Brooklyn and many other cities that take their water from the deep underground sands or mixtures of sand and earth. You will find frequently small passages of water, some of them threadlike in magnitude, almost capillary passages, and from that up to passages that flow many gallons a minute."

I have at the hotel, but forgot to bring with me, a sample of this Long Island sand, which is extremely porous, and which will run almost like hour-glass sand when poured from one thing to another. It would be impossible for any passages to be formed in such sand. They would fill up. Besides, all of the information that we have with regard to the water coming into driven wells can be accounted for without the existence of such passages. The sand grains have interstices between them, and the water filters freely through those interstices to the wells—not through large passages, but through the interstices between the sand grains. It is obvious that no one has ever been down there to find out, and there is no reason whatever to think that such passages exist in sand.

I have had occasion, in building aqueducts and in building deep trenches in connection with the building of dams and dikes, to excavate in sand to great depths, and I never saw such a passage. Professor Burr speaks of it as being of most frequent occurrence. It is a thing that I have never seen, with a great deal of experience in those lines.

So that I think that any suggestion of underground passages in sand or sandy material or earth, especially when it is under a pressure of many tons to the square foot, is not based upon any correct observation of facts.

The statement made goes on to say:

There are wells fed by such water on Long Island, and in our recent examinations for a tunnel under the Hudson River near New Hamburg we found such a passage by diamond-drill borings about 200 ft., if I recall the depth rightly, below the surface."

That might and certainly does give the suggestion that those are passages in sand. But I will call attention first to the fact that he mentions diamond-drill borings, which are only used in excavating through rock; and I also know that those borings made in New Hamburg (for I am connected with that same work) are borings into limestone, and it is common to find cavities in limestone. Those are found in the Mammoth Cave, for instance, where there are streams that flow through the limestone; and all over the country there are cavities in limestone. But it is not pertinent in the least to bring a statement of that kind here in connection with this alluvial material under the Gatun dam.

In answer to a question from Mr. Kittredge, "When was that, Mr. Burr?" Professor Burr goes on to say:

"Within three months—two months and a half ago; and those are not uncommon experiences. They are within almost, you may say, the daily range of hydraulic engineers or civil engineers engaged in that class of work."

It is a fact, if it had been so stated, that cavities in limestone are frequent; but the inference to be drawn from that is that it is a cavity in sand, and that that cavity might find its way through this half mile of dam and cause a hole through which water could flow with sufficient speed to wreck the dam, and I say that it is groundless.

In the next paragraph it is stated that "as an engineering guess, you might say that it probably would stand," but it goes on to say that there is grave danger. It seems to me it is decidedly an engineering guess that there is any possibility of passages.

SENATOR HOPKINS.—Your claim is that there is no guess about it, but that it is demonstrated to a certainty that the dam will stand as put there?

MR. STEARNS.—Absolutely; and it is ignoring of the information obtained in the last fourteen years in regard to the movement of water through sands and other earths that makes any such statement as is made here by a member of the majority an engineering guess which is a very bad one.

Reference has been made in some of the testimony to an upward flow of water through some of the pipes driven down into this geological gorge at the Gatun dam. The principal flow occurred in pipes that extended down into that 58 ft. in depth of porous material in the deepest gorge—that 58 ft. of material, or thereabouts, being under about 200 ft. of impervious material.

The height of the top of the pipe when that flow occurred, as regards the level of the water in the river is not given; but the surface of the ground at that point is 7 ft. above the surface of the river, and probably the pipe stuck up somewhat farther at the time this flow occurred. I think it is fair to say that the pipe would probably have been at least 10 ft. above the surface of the river, showing that in that pervious material at the bottom of the gorge the water was under greater pressure than that due to a height of 10 ft. above the river.

Now, the river for 10 miles up to Bohio is at sea level; consequently that water could not have come from the river, because it would not filter 10 miles underground. Water does not do that unless it has a great deal of pressure to force it through; and it is a proof of the imperviousness of this blanket of 200 ft. of material, because that water evidently came from some of the adjacent high ground, probably only in small quantities, and got under that 200-ft. blanket of impervious material and could not find its way out into the river and relieve itself—that is, 10 ft. more head or more pressure at the bottom of the gorge than that due to sea level did not force that water to relieve itself, because it was overlaid with such an impervious material.

The letter from Mr. Maltby sending these borings contained at the end a statement, after speaking of this upward flow of water, that it proves the impervious character of the 200 ft. of material above it.

That porous material, which in the report of the majority is spoken of as if it might be almost anywhere under the section of the dam, is down under this blanket, so that there can be no communication with it.

But I made one computation that, supposing (and this is a supposition or an assumption) that this impervious blanket were entirely taken off and the water had free access to that porous material at this point, and again the blanket were taken off at the lower end and the water had free access to that part, then the pressure of 85 ft. acting upon that material, if it were as porous as the most porous sand that I was able to test, or as porous as the Long Island sands, or as porous as the coarsest gravel that I had occasion to test, would let through but 2 cu. ft. of water per second. Of course that does not occur and will not occur, because the impervious blanket will stop it; but even if there were free communication, it would mean a quantity of water that could have no significance, either in the loss of water for canal purposes or in regard to the safety of the dam.

It is stated in the majority report (p. 47), among many suggestions as to the great gravity of the situation if the dams proposed by the minority should be adopted, that the Board is therefore of opinion that the existence of such costly facilities for the world's commerce should not depend upon great reservoirs held by earth embankments resting literally upon mud foundations or those of even sand and gravel. The Board is unqualifiedly of opinion that no such vast and doubtful experiment should be indulged in, but on the contrary that every feature of whatever nature should be so designed and built as to include only those features which experience has demonstrated to be positively safe and efficient.

I claim for the dams which are proposed by the minority that experience has demonstrated that they are absolutely safe and efficient, and that they have been carried in dimensions away beyond past precedent so as to make them doubly safe.

and drifted into this position; and that settled for three or four years, and went down a total of about 3 ft. But that was a great depth of mud; here it is sand and clay, which would not settle, except for some limited portions where some vegetable matter is encountered.

Of course I cannot say accurately; but I should say that it would not probably be, under that pressure, more than 2 ft. in all. I should expect that 1¾ ft. of the settlement would occur, if it occurred at all, during construction, and that it might go 3 ins. more afterwards. That is merely a guess of what might happen. I should not think there would be more than that in this place. If there was more organic matter it might be twice or three times that, but about in that proportion. That is, most of it would occur when the weight was put on it.

In some of the testimony which has appeared before the committee it has been stated that this whole matter here was an engineering guess. In fact, when the matter came up before the Board of Consulting Engineers, I think it was at least six weeks after I made my first statement to the Board before any criticisms that I recall were made unfavorable to the safety of the dam, and then it was made by only one member of the board, who did not criticise it at all in detail, but said it was an engineering guess, to which he would not subscribe his name.

I wish to say very distinctly that this is not, in my opinion, an engineering guess. It is as capable of demonstration as any problem that comes up in engineering; and further than that, that there has been allowed a factor of safety that is beyond all factors that have been allowed in the past.

SENATOR MORGAN.—Beyond all precedent?

MR. STEARNS.—Beyond all precedent, except in the case of this dam which I built, which is the precedent for this particular construction.

I am going to attempt to make this clear to laymen, and I think I can. The forces which act to move a dam of this sort from its position, to wash it away, or push it away, are, first, the pressure of the water against this mass, trying to slide it downstream.

The way in which dams have failed on some occasions—for instance, the Austin dam—is by sliding; that is, the water pressure against the dam was sufficient to cause a section of the dam to slide downstream. I do not think it needs the assistance of figures to show that that dam will not do so. That weighs about, as I recall it, 63 times as much as the pressure of the water against the upstream space. That, I think, should be obvious to a layman or anybody else—that that dam could not be pushed away as a whole.

SENATOR MORGAN.—There has been a point made there by one engineer or two that I just wanted to mention, and that was that this substratum of indurated clay was liable to compression on account of the great weight upon it.

MR. STEARNS.—As one handles that indurated clay they would call it rock; and that is what it is, a fine-grained rock. It looks something like a fine-grained whetstone, only it is softer.

For any such pressures as come here that is just as good as the hardest rock that you ever saw, and it gave the impression of being very compact.

The dam, I suppose it has been stated to you, is half a mile—2,625 ft.—through. If a dam cannot fail by the pressure of water pushing it away, as a whole, it must fail, if at all, through the water going underneath or through the dam.

At a point 374 ft. from the water line the dam is as high as the water in the lake and absolutely no water can come out at that point. And at a point halfway down the slope of the dam, the distance below the water surface is approximately about 25 ft., so that water could not come through. It would come up, if at all, away down toward the lower end of the dam, as springs.

SENATOR MORGAN.—It would come up at the point of least resistance, wherever that might be found?

MR. STEARNS.—Yes; and that would be toward the lower end of the dam.

First, I will take up the point that the material beneath the dam is tight. That, I believe, is very well shown by the borings, and by one other thing that I did not mention. I had particularly requested, in my communication to the Isthmus that they take a sample of the material pumped up from the boring hole in a pail, without letting anything spill over from the pail, so that it would contain both the fine material and the coarse material.

Those samples are taken by letting the water run into a pail, and then letting it overflow, in which case the coarse material is obtained and the fine material is not preserved. This sample was taken as I requested, and what we saw in the bottom of the pail was sand and fine material mixed with it, so that one would say it was practically impervious; and then the last thing that settled in the pail was the fine clay, which left a shiny surface over the whole thing, about as you see in the bottom of a mud puddle in the street after a rain, and that makes the bottom of the mud puddle pretty nearly water-tight. But it showed very clearly, as a single sample, that there was this fine clay in the material washed up in sufficient quantities to make it substantially, if not absolutely, water-tight.

SENATOR ANKENY.—Is that surface mica? What is that shiny surface?

MR. STEARNS.—This was all clay—finely divided clay—that settled in this pail of water.

One way of determining the safety of a dam of this sort with respect to filtration through the material underlying it is by making assumptions that instead of being impervious, that material is a pervious material; and then it becomes capable of estimation as to how much water will pass through these alluvial sections.

Suppose that, instead of the impervious material here, there was a fine-grained medium sand. I have made experiments, and they have been made by Mr. Hazen also, as to how much water will pass through such a sand, and the computations showed that if there were this medium sand, the total quantity which would pass through would be 10 cu. ft. a second.

Now, if a fine sand be assumed, the tests that I have made show that the quantity would be 2½ cu. ft. per second, and if a very fine sand, only 0.2 cu. ft. per second. That is, taking assumptions which, with the possible exception of the last one, are more favorable to filtration than this actual material that is here, then the filtration would not be an amount that would cause any upheaval at this end to interfere with the safety of the dam.

As to what experiments or tests warrant these statements, I will tell you.

Most of the experiments which have been referred to as laboratory experiments were made with a galvanized-iron tank filled with the sand or other material to be tested, the water being applied at the top, at a constant level, and drawn out at the bottom through a faucet, and with gravel underdrainage.

SENATOR KITTREDGE.—You are now speaking of the conditions under which the tests were made which I asked for a few minutes ago?

MR. STEARNS.—Yes. The pipes extended through these tanks, through the material which was being tested, at two levels, 3 ft. apart. The faucet would be closed to a considerable extent, so as to cause the water to back up in these pipes, and the pipes being at the side of the tank with a glass tube, so as to show the height to which the water would rise, one of them would show the pressure of the water at the upper point, 3 ft. above the lower one, and the other would show the pressure of the water at the lower point; and the difference in level between the water in the two tubes would represent the amount of resistance that 3 ft. of sand would offer to the passage of the quantity of water then passing; and the water passing was measured carefully. By opening or closing the faucet different quantities of water could be made to pass, and the differences in amounts of resistance caused in passing through 3 ft. was measured. It is rather a hard matter to explain.

SENATOR KITTREDGE.—When was that test made?

MR. STEARNS.—These tests that are referred to in this pamphlet were made in 1891 at the Lawrence experiment station of the Massachusetts State Board of Health, of which I was chief engineer.

Then another series of experiments were made in the same general manner under my charge, which covered a period of two years, in 1895 and 1896, I think. They were made with apparatus of this same kind and with surface soil taken from the ground, with gravel, both fine and coarse, and with sands of four different grades, the finest being very fine.

SENATOR KITTREDGE.—Where were those tests made?

MR. STEARNS.—They were made in Clinton, Mass., on the Metropolitan Water-Works, of which I was chief engineer. The tests made there confirmed those made at Lawrence. There was a question whether such tests would be applicable to the horizontal filtration, because they are made with the water going down vertically. So I had another set of tests made in a tank, a long wooden box 60 ft. long and 6 ft. wide and 8 ft. high. That was filled with the material to be tested, and the water at one end was supplied as it would be at a dam. The water was raised 8 ft. above the bottom of the box, the other end of the box being kept empty. As a result we found that the horizontal filtration of water through the materials, or a particular material tested, was the same as the vertical filtration.

The next test was made to ascertain whether materials under a great pressure would behave differently from those under a pressure of only a few feet; and the material to be tested was put in an iron pipe, about 10 ins. in diameter, and I think about 10 ft. long, and a flange was bolted on the upper and lower ends, and the pressure due to the water supply of the town of Clinton was applied to it—something like 150 ft. head. And the same laws applied under 150 ft. head acting upon 10 ft. of material as applied to these lower pressures.

You see I am trying to show that this is not an engineering guess; but that the engineering guess comes in in not applying this information. The laws which are deduced are simple, and I think can be understood. The quantity of water that will filter through sand or other material varies with the pressure. That is, the quantity that would filter underneath the Gatun Dam, if it [illegible] in this alluvial valley a porous material, is proportionate to the pressure; that is, with half as much pressure acting to force water through, there would be half as much go through, and so on. The experiments showed that it was directly proportioned to the pressure.

Now, as to the resistances: that was also found to be directly proportional to the length of filtering material upon which tests were made. That is, if the distance through this dam were only one-half as great as this total distance, then there would be only half as much resistance offered to the passage of water. I think a person would naturally agree that that would be the case, and that is what is found by tests. That is, if it filters one-half the distance, it will filter with the same head of water twice as fast.

Putting those things together in what an engineer would call a formula means that the quantity of water which will filter beneath any dam or through any other material is proportional to the water pressure divided by the length through which it filters. That is, the amount that would filter in a case like this, in comparing with another dam, is 85 ft. divided by this total length, or, leaving out a little at each end, it would mean that it was a pressure of 4 to every 100 ft. of resistance. That is because the relation of this depth to the resisting length is 4 to 100, approximately.

The fact of this 85 ft. head here is not important without one also takes into account the distance through. That is the one thing that needs to be considered for filtration, the filtration of four pressure against one hundred of resistance, and it would be the same, substantially, as a dam 100 ft. thick and with 4 ft. of water against it, or 200 ft. thick with 8 ft. of water against it. I do not know whether I have made myself clear, but the result of that experiment and of using common sense is that the quantity of water which will filter through any dam can be diminished almost at will by increasing the thickness of the dam.

To sum up what I am saying: Even if that were a porous sand of medium size there would not enough water come through here to disturb the dam; and the material is shown conclusively by the borings to have so much fine material in it that it would not filter one five-hundredth part of what medium sand would filter.

I have not attempted to go into the other laws deduced by experiments, but I will say a word or two. It is the finer particles in a sand or gravel that govern the amount of filtration, and not the average size of the whole mass. If there are 10% of fine particles, it has been found that that governs the filtration.

In the experiments which I made myself——

SENATOR KITTREDGE. You now refer to the experiments which you have already mentioned?

MR. STEARNS. Yes; at Clinton. These were the results: A soil, or surface loam taken from gravelly and sandy territory, and used for an experiment, would let through a quantity which I will call unity—1. Very fine sand, of which I had a sample and failed to bring it here, would let through 14; a fine sand, 176—that is, 176 times as much as soil, which is taken as the unit—the medium sand, which I have been using for the purpose of this illustration, let through 784 times as much as the surface soil; and the coarse sand, which was of uniform grade, let through 4,353 times as much as the surface soil.

Now, this material at Gatun is, with its fine clay, comparable with that surface soil alone, which had the fine particles of what the geologists would call rock flour in it, and also some organic matter which is in the top soil. That is, the quantity that filters varies as the square of the diameters of the finer particles, and that is the way it works out in practice.

THE CHAIRMAN. You are getting beyond our depth, now, Mr. Stearns. (Laughter.)

MR. STEARNS. I am going to say just one other thing, that you can appreciate.

I built an experimental dike in this large wooden tank that I spoke of at Clinton, and raised the water 8 ft. against that dike made of this material taken from the surface of the ground. The water that passed through that dike was so small in quantity that it would not run in a stream from the end. As I remember, it was dripping into a bottle, when I saw it, at the rate—well, I think it was 30 drops a minute.

SENATOR MORGAN. If you have come to a point in your statement where you wish to take up a new subject——

MR. STEARNS. I have still something more to say: It has been stated in the testimony of Professor Burr that dams had failed, and he said that there were numerous instances. He was asked to give one or two instances, and he mentioned the failure of the Mill River reservoir dam at Williamsburg, Mass. This dam, built in 1865, failed in 1874, at a time when the water was 4 ft. below the top, and it failed by the sloughing or caving of the lower end of the dam. A committee of the American Society of Civil Engineers was appointed and investigated the failure. They found that there was a specification which was not signed, and they could not find

The sea-level canal requires dams; and one proposition is a dam at Gamboa with a core wall of concrete extending from the rock upward at a place where the rock is 170 ft. below the water level of the proposed reservoir. That is far beyond actual precedent.

I have not any question but that such a dam can be made perfectly safe. But while saying that they should not go beyond actual precedent, the majority, in the suggestion of a core-wall dam at that place, have gone far beyond actual precedent, for I know of no core walls in existence that are more than 125 ft. high from the rock, while this is up to 170.

There is also the suggestion that that could be a solid masonry dam; but I never saw any plans drawn with reference to one for that elevation, although plans have been drawn for one of a somewhat higher elevation which may have been used as a basis for that statement, that it would cost the same.

SENATOR KITTREDGE.—May I ask just one question?

MR. STEARNS.—Certainly.

SENATOR KITTREDGE.—You say 170 ft. below water?

MR. STEARNS.—The rock at the lowest point on which the core wall would be based is 170 ft. below the proposed water level.

SENATOR KITTREDGE.—Below maximum water level—is that it?

MR. STEARNS.—That is the maximum water level; yes—the greatest height.

SENATOR KITTREDGE.—You do not mean that the foundations of this dam are 170 ft. below the surface of the water?

MR. STEARNS.—Below the surface of the water after it is raised to its highest level.

SENATOR KITTREDGE.—That is the point.

MR. STEARNS.—Certainly.

The majority report makes only a very casual reference to the dams across the Gigante and the Caño rivers, and the Gigantito.

An engineer, a member of the majority, in describing those dams in a preliminary way, suggested that they be made of earth without any core wall, and that they be carried 15 ft. instead of 50 ft. above high-water level in the reservoir; that they be made 50 ft. wide on top instead of 100, as proposed here, and that they be given slopes of one on three.

That is suggested in a place where there have been only the French surveys, where nobody has examined the sites, where the material has not been bored to find out its character nor the depth to the rock. He evidently did not have the fear of earth dams before his mind when he presented a plan of that kind.

SENATOR DRYDEN.—How deep a dam did he propose?

MR. STEARNS.—I have a note here in his handwriting—the Gigante, 55 ft.; the Gigantito, 60 ft., and the Caño, 65 ft. Those figures are substantially the same as the figures for the proposed dams at the Pacific end, which I believe some members of the majority have thought might not be sufficient.

LOCOMOTIVE REPAIRS on the Bessemer & Lake Erie Ry. during the year ending Dec. 31, 1905, averaged as follows: Light repairs, 4.46 cts. per mile run; general repairs, 1.73 cts.; extraordinary repairs, 0.63 ct.; total, 6.82 cts. per mile run. The 107 locomotives made a total engine mileage of 3,146,310, and the total cost for repairs for the year was as follows: Labor, $128,157; material, $86,490; total, $214,647.

FLAT FLOOR STEEL DUMP CARS OF 100,000 LBS. CAPACITY; ST. LOUIS & SAN FRANCISCO RY. SYSTEM.

The great advantages of a freight car which can be used either as a dump car (for handling coal, ore or similar materials), or for handling general traffic, has brought about the development of several types of flat floor cars which dump their load when desired through trap doors in the floor of the car. On Western railways particularly, such cars are preferred to the hopper-bottom type of car.

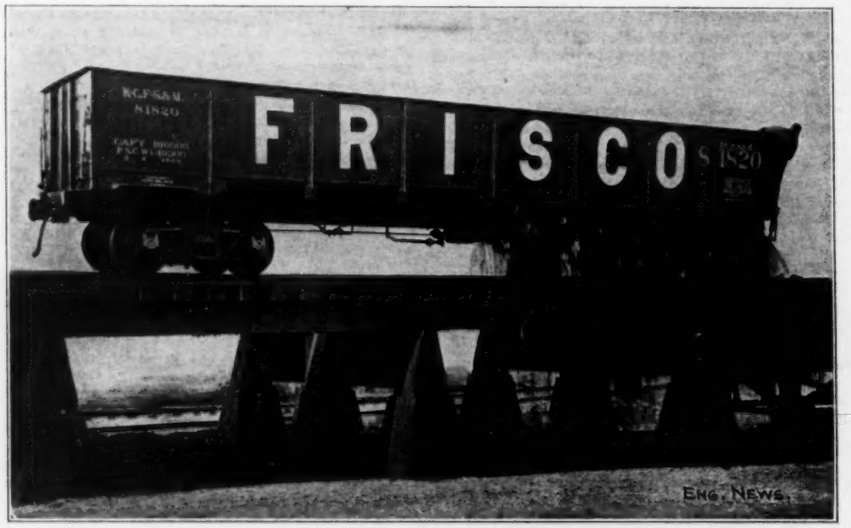

FIG. 1. GENERAL SERVICE GONDOLA CAR OF 100,000 LBS. CAPACITY DUMPING HALF ITS LOAD.

We illustrate herewith an all-steel flat-floor dump car recently designed by the Pressed Steel Car Co., which is building 200 of these cars for the "Frisco" system. The car is designed for 100,000 lbs. load, which is obtained by a car body considerably longer than the standard freight car. The car floor is made with 16 drop doors, so that practically the whole bottom of the car drops. The eight doors over the trucks drop to a clear opening of 23 ins., while the eight doors in the center of the car have a 26-in. clear opening.

The center sills are pressed to form an inverted V at the top, and are riveted to a T-shape which has a 4-in. top, and are reinforced with angles at the lower edges. Thus the center ridge of the car is made only 4 ins. wide. The body transoms are built up of plates and angles which give a 7-in. space while the cross bearers are of pressed steel shapes and present a 3-in. top surface.

The doors are operated by a so-called "creeping shaft" mechanism, which is carried in slots by the cross bearers and bolsters, and while the doors are closed by chains attached to these shafts, the latter are automatically moved over underneath the doors when fully closed and the load is thus securely supported and accidental discharge is rendered impossible.

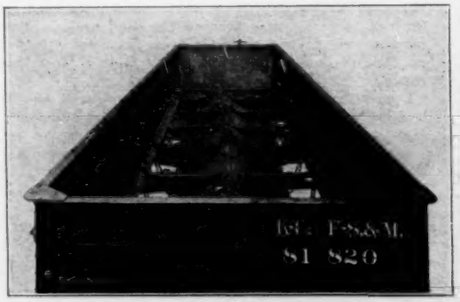

Fig. 2. Inside of General Service Gondola After Dumping.

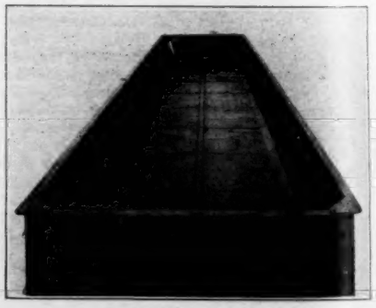

Fig. 3. General Service Gondola with Trap-Doors Closed.

An exhibition test of one of these cars was made on March 7 at the National Tube Works, McKeesport, Pa., when a car loaded with 99,300 lbs. of run-of-mine coal was dumped in the presence of a large number of railway officials.

Fig. 1 shows the car on a trestle at the moment of dumping half its load. Fig. 2 shows the interior of the car after dumping the whole load. It was estimated that the amount of coal remaining in the car after dumping was not over 600 to 700 lbs.; the greater portion of this will run out when the car is jarred to any extent, such as the coupling of an engine or slight shifting. The time required for dumping is about six seconds for each half of the car, and the time consumed in closing the doors about one-half minute for each of the four sections. Such a car can be placed, dumped, and the doors closed to form a flat bottom gondola car by two workmen in four minutes or less.

Fig. 3 shows the interior of the empty car with the trap doors closed.

The general dimensions of the car are as follows:

	ft.	ins.
Length over end sills	42	9
Length inside of body	41	9
Width over stakes	10	2
Width inside	9	6¾
Depth of body to top of sides	4	4
Length of doors in clear	4	10
Width of doors in clear	4	2¼
Height of floor from top of rail	4	5
Truck centers	31	0

THE NEW ELECTRIC CARS of the Chicago City Ry. are of special design, with long platforms. The platforms are divided by a bar with the intention of inducing persons to enter at the part next the dashboard and to get off at the part next to the body, but it is found almost impossible to educate people to follow this plan. The cars are 45 ft. 9 ins. long over all, 9 ft. wide outside, and 11 ft. 10 ins. high from the rail to the trolley board. They weigh about 26 tons, and cost about $5,500 complete. The outside panels are of steel sheets, so that in case of damage by wagons, etc., the injured piece can be quickly replaced without having to repaint the entire side and keep the car in the shops. At each end are two longitudinal seats, while in the central part are 14 reversible transverse seats; the total seating capacity is for 44 passengers. There are double windows for winter use, but for summer use the windows are dropped into pockets in the side and the end doors are removed, protection being afforded by the vestibules. Electric heaters are used, under the seats, and were adopted as being less troublesome and more economical than hot water heating. The cost is estimated at 73 cts. per car per day of heating season, as compared with 80 cts. for hot water. Three separate degrees of heating are obobtainable by means of a switch, this being adjusted in accordance with the outdoor temperature. Each car has four motors of 40 HP., two to each truck. The schedule speed is about 9½ miles an hour, with from 5 to 9 stops per mile. At first no straps were provided for the use of standing passengers, as it was thought that they could more conveniently take hold of the handles on the backs of the cross seats. But it was found that the standing passengers tended to crowd at the ends, and overhead straps have therefore been fitted opposite the longitudinal seats. The trucks, bodies and motors were purchased separately, and assembled at the company's shops.

A STEAM CANAL BOAT is to be experimented with by the Ohio Boat Co., of Lockland, O., whose seven boats with gasoline and oil engines were described in our issue of March 22. The company has eight steel canal boats, which have been in service for nearly two years. One of them has not yet been equipped with power, but it is now proposed to install a steam engine with propeller.

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ADVERTISING

"Contract": Rates furnished on application.
"For Sale": $1 50 per inch.
"Proposal": $2.40 per inch.
"Want": See pages 26 and 27.

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We do not ordinarily look to Tammany Hall for political or economic wisdom, but, thanks to Mr. Bourke Cockran, it has just put on record some most sound doctrine regarding municipal ownership and the charges for services rendered thereunder, as follows:

> Whether a public utility be administered by Government itself, or by a private corporation, we hold that every person who uses it should be held to pay the cost of the service which he receives, and that no man should be compelled by taxation or otherwise to bear any part of his neighbor's expenses for light and power, for telephone service, or for transportation of himself, or of his goods.

The foregoing proposition involves far more than appears on its face. To put it into effect, in the case of many existing municipal utilities, would require a complete readjustment of the burden of service-cost between (1) public and private users and (2) individual private users. As matters now stand it frequently happens that private consumers of water and light are bearing an undue share, and in the case of water the whole cost, of the public service, whereas the cost of water for fire protection, sewer flushing, street sprinkling, and public buildings, and also lights for streets, parks and public buildings, should be borne by the taxpayers at large. And since every tub should stand on its own bottom, the street, sewer, police, fire and school department should each be charged for the water and light furnished it by the corresponding departments. The whole burden of the water and light service having been apportioned between the public and private consumers, and properly distributed among the various municipal departments, the matter of equitable charges to private consumers is simplified, and at the same time these charges, in many instances, would be materially lessened. It is almost universally admitted now that gas and electric light should and must be sold by meter measurement. The reasons for basing private water charges on meter measurement are in many respects even stronger, and eventually they are bound to prevail. By such means, only, will every man pay for the water he uses, instead of shifting its cost, as indicated in the quotation, upon his neighbor. A readjustment of public service charges along the lines indicated is greatly needed in some cities where private ownership prevails, and where the water or light companies, following the lines of least resistance, are charging private consumers too high and the general public too low rates in some instances and reversing the inequality in others. Under either municipal or corporate ownership, the taxpayer and the consumer alike have rights and duties that should not be ignored. The sooner this is learned, the sooner will universal justice prevail and the less need will there be for that fear of Socialism which is a nightmare to many who are quite indifferent to the various economic and moral wrongs that are masquerading under other names, but are none the less subversive of good municipal government and a square deal to all.

THE MULTIPLICATION OF STATE COMMISSIONS DEALING WITH WATER AND SEWERAGE MATTERS.

The proposal for a joint metropolitan sewerage commission for New York and New Jersey, made by the New York Bay Pollution Commission, and the conference on the pollution of the Delaware River held by representatives of the States of New Jersey and Pennsylvania, resulting in a proposal for joint action by those two States and by New York to prevent the pollution of the Delaware, all as noted in our issue of March 22, suggest a brief consideration of the governmental machinery already existing in the several States to deal with questions of water pollution. New York now has a State Department of Health, a State Water Supply Commission (both of which are permanent), and for two years past there has been in existence the New York Bay Pollution Commission. New Jersey has a State Board of Health, a State Sewerage Commission and a Passaic Valley Sewerage Commission, each of which is a permanent body. Pennsylvania has a State Department of Health and a State Water Supply Commission. The question naturally arises whether this machinery is not more than adequate to deal with water pollution questions. By saying "more than adequate," we mean to suggest that it might be far better to consolidate some of the existing commissions in the several States rather than to provide a new one, and after such consolidation was effected to refer all matters of interstate pollution to the single strong commission that might be hoped for in each State, thus securing such joint action as might be necessary.

The proposal recently made by the New York Bay Pollution Commission does not presuppose the creation of any new body, of necessity, in New Jersey; but it does contemplate the continuation of a third commission in New York State. Is this necessary or desirable? There may be local conditions in each of the States named requiring the three separate commissions in New York and New Jersey and the two commissions in Pennsylvania, but on broad general grounds a reduction in number is certainly desirable. In Massachusetts nearly all the matters entrusted to the several commissions noted above have been most successfully handled by the State Board of Health. We think that in the case of New York and New Jersey, at least, it would have been better to have strengthened the State health departments rather than to have created one, at least, of the additional commissions. There were, however, some special reasons for the course pursued. In New Jersey there was some justification for establishing the Passaic Valley Sewerage Commission, on account of the fact that large construction work was necessary in the Passaic Valley. It might have been well, however, to have deferred the appointment of a commission for construction work until the problem of final disposal had been more thoroughly worked out. As matters stand, the New Jersey State Sewerage Commission and the Passaic Valley Sewerage Commission have been costing the people of New Jersey some $20,000 a year or more, and thus far have accomplished comparatively little. A small part of this sum entrusted to a properly organized and equipped State Board of Health each year, it seems to us, might have yielded far better and more permanent results. There was much to justify the creation of a separate commission for the study of the pollution of New York Bay, but in this case, also, assuming a properly organized and equipped State Department of Health, it is questionable whether the work might not better have been entrusted to the latter. The State water supply commissions of both New York and Pennsylvania were created in large part to deal with matters not properly pertaining to health departments. The New York Commission adjusts conflicting interests between different communities seeking to draw water from the same drainage areas, and to it is also entrusted the matter of seeing that water and land interests are properly protected when a city reaches out into rural districts for a supply. Thus far the Pennsylvania Commission has seemed to do little work except to pass upon applications for charters. The New York Water Supply Commission has mapped out a very ambitious program of investigation of both the water supplies and sewerage systems of the entire State. To a large extent this program covers the very field that seems naturally to pertain to the State Board of Health.

The New York State Health Department, as shown by its last annual report, is very poorly equipped for its work, owing to limited appropriations. This applies particularly to its investigations of water supply and sewerage questons. It seems fair to ask whether the State Health Department of New York is not being starved to provide funds for the other commissions. We do not suppose that this is actually the case, but the overlapping of work must make it difficult for the several commissions to secure necessary appropriations. Certain it is that in Massachusetts, where all the sanitary work naturally falling to a State Board of Health has been continuously entrusted to such a board, highly gratifying results have been secured. In several other States, where the experiment of splitting up the work has been tried long enough to see how it works, the results have been in many respects disappointing. Connecticut, some will remember, launched a State sewerage commission a few years ago, but after doing some good work with a very limited appropriation it came to an untimely end. If the small sum of money appropriated to it had been given to the State Board of Health, it seems very likely that the appropriation might have been continued to the present time, and perhaps increased slightly from year to year. About the same time that Connecticut split up its work of sanitary investigation, Ohio entrusted to its State Board of Health an investigation of the sanitary condition of the waters of the State. This work has increased from year to year, both in amount and efficiency, until Ohio is one of the leading States in the Union in its supervision of public water supplies and sewage disposal. Only recently an appropriation of $15,000 has been made for an investigation by the State Board of Health of the actual workings of water and sewage purification plants within the State. Here, as in Massachusetts, concentrating sanitary investigations in the State Board of Health has worked admirably.

We are well aware that many troublesome questions are raised by the foregoing discussion, or would be, if those who know the facts spoke frankly; but is it not high time that they were raised and, if possible, settled? The questions center around the competence of certain State boards of health in the past; the degree of confidence with which their work and their personnel has inspired the public; the proper domain of a State Board of Health, and the persistent tendency of all governments to appoint new commissions and enact new laws instead of making the most of existing commissions and laws.

THE FOUNDATION FOR EARTH DAMS.

The drawings and description of the Gatun dam, as planned by the minority members of the Board of Consulting Engineers for the Panama Canal (and shown in our issues of Feb. 22 and March 1) appear to us absolutely conclusive as to the stability and safety of this structure. In the recent testimony before the Senate Committee, however, the design proposed for this dam has been severely criticised by Prof. Wm. H. Burr, one of the "sea-level" members of the Board, and the criticisms have been answered by Mr. F. P. Stearns in giving his testimony later. We print elsewhere in this issue the substance of the testimony of both these gentlemen relating to the Gatun dam, and are sure that all our readers

interested in this class of engineering work will find it of great interest.

We may separate Mr. Burr's criticisms of the Gatun dam into two divisions—the fair and the unfair. In the latter class we should place his references to the Mill River dam failure, the leakage of a reservoir at Brooklyn, and the finding of rock cavities in a deep diamond drill boring. Such evidence may possibly mislead a committee of laymen; but to an engineer it is obvious that none of these things have any real bearing on the safety of the Gatun dam. Such evidence against the Gatun dam is as unfair and misleading as it would be to present as arguments against the sea-level plan the frequent crevasses in the Mississippi levees, the failure of the Bouzey masonry dam in France, and the destruction of the old Schuylkill canal by the floods in the river alongside.

An engineer acting as a professional expert adviser ought never to present before laymen evidence that really has no bearing on the case, or that he would not himself take into consideration in forming an opinion on the question at issue. We can but think that Professor Burr's references to the Mill River dam failure and his other testimony which followed were made on the spur of the moment, in the course of the examination, without deliberate intention of misleading the Senate Committee and without realization of the judicial position which every member of the Board of Consulting Engineers ought to hold in this important matter.

Eliminating now from Professor Burr's testimony those irrelevant matters, which are to be classed with a lawyer's appeals to the prejudices of a jury, it appears that Professor Burr really objects to the Gatun dam because of the more or less permeable material beneath its base. As our readers will recall, the Gatun dam is to be a great artificial hill, built across the valley of the Chagres River and forming a vast mound of earth a mile and a half long, half a mile wide and 135 ft. high. For most of its length the hardpan or indurated clay lies at shallow depth beneath it, and it will be easy to excavate a trench beneath the dam to this hardpan. At two places, however, the line of the dam crosses old geological gorges which have been filled with alluvial materials. These materials, as may be seen from the borings, on p. 206 of our issue of Feb. 22, are sand and clay, with rarely a little gravel and no boulders. In the lowest part of these gorges the material appears to be more permeable than in the upper part; but the upper 200 ft. is considered by Mr. Stearns (who is unquestionably the engineer of largest experience and widest reputation in this special field in the United States, and, in fact, in the world) to be practically impervious to water. Mr. Burr, on the other hand, claims to see with his mind's eye—since he cannot see them with anything else—all sorts of cavities in this material. He ignores entirely the accumulated facts and experience regarding the resistance to the passage of water through permeable materials; and because engineers of an earlier day supposed it to be necessary to carry dam foundations always to rock or hardpan, he characterizes a design which contemplates any other course as an "engineering guess."

We need not take space here to summarize Mr. Stearns' excellent and complete answer to this criticism. Instead we will venture to attack the Gatun dam problem from another point of view.

Suppose we state the problem in this way: Given, a river valley across which an earth dam is to be carried. In building this dam the surface soil is removed until a layer of fairly compact earth is reached and the dam is built up directly on this. The dam itself is made impervious to water by proper construction, is carried so high that overtopping is impossible, is suitably protected from the action of waves and rain, and is made of a breadth 25 times as great as the height of water confined behind it. It is now desired that a large part of the water flowing down the river *shall pass beneath the dam.* What materials underneath the dam would make this possible?

It has been fully proven that the resistance to flow through sands, and even through gravel, is too great to permit the passage of any considerable amount of water under the head named; hence the only way for the water to pass beneath the supposed dam would be by having beneath it a very thick and broad stratum of clean boulders and pebbles. Suppose that such a stratum exists, deep down under the dam, and, further, that it turns upward somewhere above the dam and communicates with the bed of the river, while below the dam it also rises to give the water exit. We shall under such conditions have some of the river water entering this boulder stratum, flowing underground and reappearing as springs at some point, perhaps miles away, on the lower side of the dam. Under the conditions of head and slope named, however, the "hydraulic grade" of the water flowing through this boulder strata would be at the steepest only 4 ft. per 100, and probably very much less. Water cannot flow through the crooked and irregular channels in such underground boulder strata under that small head with any considerable velocity.

If a river of perfectly clean water, flowing over a clean boulder-paved bed were under consideration, it is conceivable that we might pass its flow, or a portion of it, underneath our dam through our hypothetical boulder layer indefinitely; but such rivers are not found in nature. The first flood in even a clear water stream brings down water heavily laden with sand, silt, mud and debris, which is at once carried into the boulder strata where our river is supposed to be flowing; the interstices between the boulders are promptly choked up and the flow beneath the dam is cut off. Each succeeding flood brings down more muddy water, which adds to the thickness of the water-tight layer covering the bottom of the reservoir; and cuts off any percolation down to the boulder stratum that passes beneath the dam. It appears, therefore, that it is *impossible* to dam up a stream under the conditions named and discharge the waters through a permeable stratum underneath the dam.

The problem above discussed is directly applicable to the conditions at the Gatun dam. There is no boulder stratum at the site of that structure, as far as known; but even if one did exist, far down at the bottom of the geological gorge, it would be of no importance to the safety of the structure. If water did pass down to it for a short time while the reservoir was filling, the passages leading to it would be promptly choked by the mud and sediment of the lower Chagres, and not all the artifice or devices of man could force any different result.

We do not see, therefore, that it makes any particular difference to the safety and stability of the Gatun dam whether the sand and clay in those old geological gorges beneath it is water-tight, as Mr. Stearns claims, or permeable, as Mr. Burr claims. If it were all an open bed of river gravel, the flow through it would be choked by the silt and mud of the Chagres as soon as the reservoir began to fill.

The illustration we have cited above is not mere hypothesis; it is actually closely parallel to what is always done in building a common sand filtration plant. Water is confined by an earth embankment or dam, under which the flow is carried by a permeable stratum or piping, branching to smaller piping and thence to stone drains and to a careful gradation of smaller and smaller sizes of gravel and sand to the surface of the filter. While the work is designed in this way for another purpose, it is actually the best plan that could be devised to cause water to flow permanently through the whole bottom of a reservoir and thence through subterranean channels beneath its walls. But we know that even with this most carefully planned and built reservoir, and with only clear water flowing into it, the bottom of the filter or reservoir is gradually choked until the flow ceases, and regular loosening of the surface and washing of the clogged sand is necessary to make the flow continue.

The position taken by the Gatun dam critics is that the waters of the Chagres River, held back by the dam, are likely to leak through the reservoir bottom and escape to the sea through underground channels. We maintain that this is pure theory, unsupported by a single specific fact. Engineers have built thousands of dams across the valley of natural streams and it is universal experience that even where some seepage occurs at first, the accumulation of silt soon makes the reservoir bottom watertight. The one and only condition where there is any possibility of the loss of the water from such a reservoir through subterranean channels is when the reservoir is underlaid by limestone, which may contain actual caverns. This condition does not exist in the Chagres Valley.

It will be understood in the above, of course, that we are not arguing against proper and reasonable precautions in dam construction. In many locations it is better and more economical to carry a dam to bedrock or hardpan than it would be to leave permeable materials beneath it and extend its width to secure watertightness and safety. There are many locations, however, where the engineer is called on to confine water by earth embankments, and it is impossible to go to bedrock with the foundation. The irrigation dams and embankments in India and our own Mississippi River levees are good illustrations. It is well, therefore, that the laws relating to such structures should be widely understood; and it should be made clear that it is not at all beyond the power of the engineer to secure safety and stability under such circumstances.

LETTERS TO THE EDITOR

The Over-Registration of Two 4-in. Water Meters on Test.

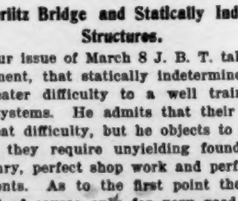
Sir: The writer being interested in the testing of water meters would like to give the following in reply to the account of the test of a 4-in. meter made by S. H. Lea, of Sturgis, S. Dak., as given in your issue of March 15, 1906. The writer believes the meter in question was fast when installed. We have found it necessary to test every meter as it comes from the factory for our own protection; one, in particular, that had the wrong change gear when received so that it registered 11% fast. It would be more satisfactory to all concerned if the factories would test water meters with the dial and train on the meter where it is to operate.

C. E. Inman,
Supt., The Warren Water & Light Co.
Warren, O., March 19, 1906.

Proper Names for Land Subdivisions.

Sir: Will some of your readers kindly inform me as to the proper names which should be used to designate the several different subdivisional corners of an ordinary government section of land. With the accompanying diagram I submit two examples: Is either method correct, or both wrong, and why?

A B D E represents the four section corners. Are the names of the subdivisional corners: (1) a b C d e quarter



section corners, and 1 to 12, including c c c c, quarter quarter section corners, or (2) a b d e half section corners. C the quarter section corner, 1 to 12 half quarter section corners, and c c c c the quarter quarter section corners, the latter also known as ⅛ cor. and 1/16 cor.

Respectfully, Chas. H. Gillett.
Appleton, Wis., March 20, 1906.

The Austerlitz Bridge and Statically Indeterminate Structures.

Sir: In your issue of March 8 J. B. T. takes exception to my statement, that statically indetermined structures offer no greater difficulty to a well trained engineer than other systems. He admits that their computation offers no great difficulty, but he objects to them on the ground that they require unyielding foundations, non-elastic masonry, perfect shop work and perfect engineering instruments. As to the first point they are to be recommended of course only for very good foundations, preferably rock, in which case there is no difficulty with elastic masonry at all. As to the matter of shop work, instruments and theory I need only say that these problems having been successfully solved by European en-

gineers should offer no greater difficulties for American engineers.

The figures 142.3 ins. should of course be 142.3 millimeters.

Yours very truly,
K. A. Müllenhoff.

Wilkinsburg, Pa., March 17, 1906.

Removing a Transit Stuck on Its Tripod.

Sir: I note an article on "How to Remove an Instrument Stuck on Its Tripod." I have never adopted the method mentioned, neither have I ever used the cold chisel or ax, but I have always used a heavy strap, which never fails to do the work. The strap is placed around the milled head of the instrument and is drawn tight until the head unscrews. I keep the strap in my office for that purpose.

Yours truly,
William S. Logan.

Arlington, N. J., March 20, 1906.

Sir: I note in your last issue a method of removing a transit from tripod when stuck fast. I have found difficulty from that source where the lower part of the instrument surrounds the tripod head. Having taken the instrument from a warm room and the tripod from a cold place, when it was desired to remove the instrument, I found it impossible to stir it. By pouring a little hot water over the part, however, and turning the transit, before the heat had reached the tripod head, I was able to loosen it without difficulty. I see no reason why this method would not work as well if the instrument became stuck fast for other reasons.

E. F. Musson.

Norwich, N. Y., March 20, 1906.

The Comparative Merits of Different Street Pavements.

Sir: The table showing comparative values of pavements used in Holyoke, Mass., as determined by T. J. MacCarthy, C. E., published in your issue of March 1, 1906, is, as you suggest, of interest. The enclosed diagram [not reproduced—Ed.] prepared by the writer shows the results given in the last four columns of this table, except, in the next to the last column, for slipperiness, he has recalculated the items, putting wood block as the most slippery pavement and macadam pavement as the least, which, he thinks, was Mr. MacCarthy's intention, though the reverse is shown. The writer's experience in laying and observing different classes of pavement in New York City during a period of 17 years, leads him to differ with Mr. MacCarthy in some of his deductions, though somewhat different climatic conditions in the two cities may account for this in some degree.

As to safe maximum grades: 10% is *desirable* for macadam and granite block pavements, but macadam is safe on any grade on which a steam roller can work, say 20%; and granite blocks, if laid normal to the road surface, present an edge which successfully prevents horses from slipping at a grade of 18%. Safe maximum grades for vitrified bricks of 8%; asphalt blocks of 4½%; sheet asphalt of 3%, and wood blocks at 2% have been found advisable.

As to percentage of slipperiness with wood block pavement at 100%, the changes suggested in this table are: Sheet asphalt at 90%, asphalt blocks at 60%, vitrified bricks at 50%, granite blocks at 33% and macadam at 20%.

As to cleanliness, it is believed that asphalt blocks should be placed with vitrified bricks at 80%.

Macadam, unless new, is one of our noisiest pavements even when in good repair, and in the last table should be placed at 40%.

Repairs to this latter class of pavement in the Borough of Richmond, based on experience, with about 200 miles during the past 10 years, cost about 10 to 11 cts. per sq. yd. per annum.

The writer knows of a granite block pavement laid on concrete foundation in Broadway, New York, under his supervision, to be so badly cut up and worn in one year as to be unfit for use. This would tend to show the impossibility of a just comparison of the life of pavements or of their repair in different cities owing to the varying condition of traffic and of climate.

Very truly yours,
Theodor S. Oxholm, M. Am. Soc. C. E.

Borough of Richmond, New York City, March 16, 1906.

Notes and Queries.

On p. 326 of our last issue the titles to Figs. 5 and 6 were accidentally transposed.

W. E. J., Knoxville, Tenn., writes:

Will you kindly advise if there is any remedy to remove blue print stain on profile paper or tracing cloth caused from developing in the rain.

A correspondent asks if any one has had experience with reinforced concrete as an unprotected roof covering. By this is meant a roof laid exactly as a floor or sidewalk would be and not covered with any roofing material. If any of our readers can furnish information bearing on this question we should be glad to publish it.

In the article on "The Design of Concrete Steel Beams and Slabs," by Mr. Edward Godfrey, published in our issue of March 15, correction should be made as follows:

In the second paragraph from the last, where it is stated that "some saving of steel could be effected by placing the rods nearer to the bottom of the beam, etc.," the last words, instead of being "neutral axis" should be "center of compression in the concrete."

ANNUAL CONVENTION OF THE AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

The seventh annual meeting of this association was held March 20, 21 and 22, at the Auditorium Hotel, Chicago. The attendance was large, over 200 members being registered. In marked contrast to some former meetings, the scheduled program was followed out in its entirety and every report was presented in due course. This necessarily implies that discussions were relatively limited; but in no case had the discussions to be cut short for lack of time. One good plan which helped materially in this respect was the elimination of any discussion on the definitions presented in the various reports, and which at former meetings have received an amount of attention disproportionate to their importance. This matter was discussed at some length in connection with the report of the 1905 meeting, and the opinions then expressed (Engineering News, March 30, 1905) still hold good.

There are 17 committees in all, and 16 of these had printed reports; 15 of the reports had been distributed prior to the convention, and the other was distributed during the meeting. The remaining committee (Economics of Railway Location) made a verbal report of progress. There were no entertainments or excursions, but the annual dinner was held as usual on the second evening, March 21. As an auxiliary to the convention there was the usual extensive exhibit of machinery, appliances and supplies in the parlors of the hotel. Abstracts of some of the committee reports will be found on pp. 340, 341 of this issue.

The meeting opened on the morning of March 20, and with a short address by the President, Mr. H. G. Kelley (M. & St. L. Ry.) The address called attention to the fact that the recent progress toward the partial electrification of steam railways indicates that the new conditions arising will in the immediate future require the attention of the members of this association, and are opening up a new and broad field for original thought and work.

The reports of the secretary (L. C. Fritch, Ill. C. Ry.) and the treasurer (W. S. Dawley, C. & E. Ill. Ry.) showed results which are summarized below:

Balance, cash in hand, March, 1905	$8,560
Receipts during the year	11,406
Expenditures during the year	10,017
Balance to credit	1,389
Balance, cash in hand, March, 1906..	9,949
Membership in March, 1905..	463
Members admitted during the year	102
Members withdrawn	9
Members dropped for non-payment of dues	27
Members deceased	1
Members lost, total	37
Net increase	65
Membership in March, 1906..	528

TIES.

This committee's report is abstracted on p. 340. In the discussion, Mr. Wendt stated that the Pittsburg & Lake Erie Ry. has used copper dating nails, but finds that they are stolen from the ties; the Pennsylvania R. R. has had the same experience. The trouble with galvanized nails was their corrosion, but he thought this was due to the date figure being raised or in relief. He thought a well galvanized nail having the date figure sunk in the head would give good service. The size of tie recommended by the committee as standard was criticised somewhat severely, the 8-in. width and 8-ft. length being considered too small. The increase in thickness from 6 ins. to 7 ins., however, was approved. The committee, however, considered 8 ft. sufficient for a well-ballasted track. Mr. McGuigan (Grand Trunk Ry.) remarked that while a "standard" size may be specified, in many cases the railway is glad to get ties of any size, and he instanced one railway as using ties 4 ins. wide. This part of the report was referred back to the committee, with instructions to specify dimensions of ties for three classes of track. The supplementary part of the report, dealing with specifications for preservative treatment, was only briefly discussed, but it was pointed out that while these provided for a preliminary steaming in the cylinder there is now a growing objection to this on the ground of injury to the wood, and some processes now omit this steaming entirely.

BALLASTING.

This committee's report is abstracted on p. 340. The discussion centered around the form of crown to be given to the subgrade to secure drainage. The committee was instructed to prepare new cross sections showing the roadbed level instead of crowned, and with the ballast level with top of tie at center in all cases; also to prepare a new cross section for stone and slag ballast. Mr. Trimble (Penna. Lines) stated that investigation showed that crowning did not last, the ballast being gradually driven down into the roadbed. As to burned clay ballast, it was voted that its absorption should not exceed 15% by weight.

YARDS AND TERMINALS.

This committee referred to the extensive work of reconstruction and improvement of yards during the past few years. The report was devoted almost entirely to the one subject of gravity switching yard on the summit or "hump" system, and its substance was given on p. 340 of our last week's issue. Mr. A. W. Johnston (Penna. Lines) stated that the claim departments assert that an increase in damage claims is partly due to the increase in the use of "hump" yards. Mr. Tratman, however, stated that the committee based its opinion in favor of such yards on the statements made by officers in charge of a number of "hump" yards.

IRON AND STEEL STRUCTURES.

This committee presented complete general specifications for the design, material and workmanship for steel railway bridges, being a revision of the specifications submitted and discussed at considerable length at the annual meeting of 1905. The 1905 and 1906 specifications were printed on opposite pages so as to show the various changes more clearly. The impact formula was objected to last year, when the report was referred back to the committee for further consideration, and is now given as follows:

The dynamic increment of the live load shall be added to the maximum computed live load strains and shall be determined by the formula $I = S \frac{300}{L + 300}$

where

I = impact to be added to the live load strains.
S = computed maximum live load strain.
L = loaded length of track in feet producing the maximum strain in the member. For bridges carrying more than one track, the aggregate length of all tracks producing the strain shall be used.

Impact shall not be added to strains produced by longitudinal centrifugal and lateral or wind forces.

A minority report signed by three of the 14 members of the committee expressed disapproval of certain items of the report, but more especially in regard to the impact formula, and some of the remarks in regard to this formula are given below:

We cannot approve the impact formula as submitted, involving the use of a single unit stress for both static and live loads. The method of proportioning bridges, which was based on the assumption that the moving load has twice the effect on a structure of an equal amount of static load is so generally in use by prominent engineers that it is to our minds unreasonable to infer that it is incorrect and bad practice.

The impact formula proposed by the majority members, before being adopted finally and promulgated to the profession as the only one worthy of use, should be either mathematically or experimentally proven. This we claim has not been done, and it certainly has not been done to our satisfaction at least. The proposed formula does not adequately account for the effect of heavy static loads involved by the use of ballasted floors, or other causes.

We believe that the impact question is one which should not be settled at the present time. The committee, as a whole, is prepared to recommend to the association the carrying out of impact tests, and this in itself is proof that although a recommendation has been made for an impact formula, the question is still unsettled, even in the minds of the committee. The minority members urge that these tests ought to be made, and believe that a reasonable impact formula can be determined from such experiments.

The committee submitted a number of changes from the specifications as printed in its report, and the discussion occupied the evening session

of three hours; the greater part of the time was devoted to the chemical and physical properties of structural steel and the question of impact. As a result of this, the committee agreed to reduce the phosphorous content in acid steel from 0.08 to 0.06%, provided that the American Society of Testing Materials takes similar action. The committee recommended an ultimate tensile strength of 60,000 lbs. (with variation of 4,000 lbs. either way). Mr. Condron wished to raise this to 62,000 lbs. Mr. Schaub, however, while not advocating a change, thought that if it was to be made it should be to a much higher figure than 62,000 lbs., as manufacturers would furnish practically the same material for a 62,000-lb. as for a 60,000-lb. specification. Reference was made to the following statement by Mr. Theodore Cooper in a discussion printed with the committee's report:

It has appeared to me that, behind the general desire for a common specification, there are two especially dangerous features, tending to degrade all the standards: First, for unanimity, it will be necessary to lower the standards so that the poorer or less progressive manufacturers can be placated. Second, lower the standards, so that the perfunctory inspector or inexperienced engineer will not be bothered to use any judgment.

The report was referred back to the committee for further investigation, and it was decided to take steps to have actual tests made in regard to the disputed subject of impact.

CLASSIFICATION OF TRACK.

This special committee presented a brief report, accompanied by a voluminous tabulation of the engine loads and traffic conditions of a large number railway divisions. It recommended the classification given below, which was accepted and will be submitted to letter ballot.

CLASS A.—All districts of a railway having more than one main track; or those districts having a single main track with a traffic that equals or exceeds the following: 150,000 freight car mileage per year per mile, or 10,000 passenger car mileage per year per mile with maximum speed of 50 miles an hour for passenger trains.

CLASS B.—All districts of a railway having a single main track with a traffic that is less than the minimum for Class A, but equals or exceeds the following: 50,000 freight-car mileage per year per mile, a 5,000 passenger-car mileage per year per mile with a maximum speed of 40 miles per hour for passenger trains.

CLASS C.—All districts of a railway not meeting the traffic requirements for Classes A or B.

ROADWAY.

This committee presented a lengthy report upon the subject of overhaul, consisting mainly of the individual views and opinions of members. It appeared that owing to the diverse views as to methods of computing overhaul and defining the manner of payment, it was not possible to submit definite conclusions. Since the report was issued, however, the committee had formulated an overhaul clause for specifications for grading, and this will be submitted to letter ballot. The clause is as follows:

No payment will be made for hauling material when the length of haul does not exceed the limit of free haul, which shall be — — ft.

The limits of free haul shall be determined by fixing on the profile two points, one in excavation and the other in embankment, such that the distance between them equals the specified free haul limit and the included quantities of excavation and embankment balance. All haul on material beyond this free haul limit will be estimated and paid for on the following basis of computation:

All material within this limit of free haul will be eliminated from further consideration.

The distance between the center of gravity of the remaining mass of excavation and center of gravity of the resulting embankment, less the limit of free haul as above described, shall be the length of overhaul, and the compensation to be rendered therefor will be determined by multiplying the yardage in the remaining mass as above described by the length of the overhaul. Payment for the same will be by units of 1 cu. yd. hauled 100 ft.

When material is obtained from borrow pits along the embankment and runways are constructed, the haul shall be determined by the distance the team necessarily travels. The overhaul and material thus hauled shall be determined by multiplying the yardage so hauled by half the round distance made by the team, less the free haul distance. The runways will be established by the engineer.

SIGNS, FENCES AND CATTLE GUARDS.

This report is abstracted on p. 350 of this issue. The committee presented specifications for wire fences with wooden posts, which were adopted. In regard to the clause requiring galvanized fencing to be manufactured in such a way as not to remove the galvanizing or impair the strength of the wire, Mr. Macdonald (N., C. & St. L. Ry.) thought it would be impossible to do this without a galvanizing treatment following the process of manufacture. Mr. Trimble (Penna. Lines) disapproved of galvanized wire fencing, and stated that the best he had seen lasted only two years.

SIGNALING AND INTERLOCKING.

This committee presented specifications for telegraph block signal work (submitted but not acted upon in 1905) and for mechanical interlocking work, the latter being the specifications adopted by the Railway Signal Association. The principal conclusion presented in the report was as follows:

The lock-and-block with track-circuit control (using intermediate automatic signals for following movements and abandoning the use of orders) should be employed instead of the system of using automatic signals for both following and opposing movements, as an adjunct to the order system for single track work.

The committee took a letter ballot during the year as to the advisability of using an upward instead of downward inclination of semaphore arms to indicate "caution" and "clear." Of 125 replies, 72 were opposed to this arrangement and 53 favored it.

RAIL.

This committee's report was distributed and taken up for discussion at the close of the afternoon meeting of the second day, and was largely of a tentative character. This is due to the fact that the proposed specifications of the American Society of Civil Engineers will be taken up at the annual convention of that society in June, and it may be deemed best to adopt these specifications, which the committee considers as in general preferable to those of the association. Mr. Abbott (Lackawanna Steel Co.) objected to specifying 25% to be cut from the ingot to prevent piping; in many cases less than this is sufficient, and there is no need for the extra waste. He also strongly objected to reducing the camber from the 5 ins. in the Am. Soc. C. E. specifications to 3 ins., as recommended by the committee. He argued that as a matter of fact 3 ins. is rarely exceeded, and that even when it is exceeded the rails can be made as good as any other. Mr. Trimble (Penna. Lines) thought that Mr. Abbott's statement was a sufficient argument for adopting the 3-in. limit, and it was adopted by vote of the association.

MASONRY.

This committee's report was presented on the third day of the convention, after the conclusion of the discussion on rails. It called attention to the failures of concrete construction (plain and reinforced), due to: (1) Improper design, (2) Premature or improper loading, or (3) Improper materials, proportions or workmanship. It requested members to call the attention of the committee to any such failures and to furnish all the information available. With the report was a set of proposed specifications for stone masonry. The report also gave some information in regard to the work of the Joint Committee on Concrete and Reinforced Structures, which has so far been of a preliminary character. Tests under the committee's direction have been made in the laboratories of about 12 engineering schools.

The specifications for stone masonry were discussed section by section, and (with a few modifications) were adopted.

BUILDINGS.

This committee's report is abstracted on p. 350. The report indirectly advocated the roofing of the space occupied by the turntable, but the discussion showed that where this has been done it injuriously affects the ventilation and light to a marked degree. It also appeared that rectangular engine houses have advantages which were not presented in the report.

TRACK.

The principal feature of the report of this committee was that dealing with easement curves, presenting formulas and instructions for laying out such curves. The committee also recommended that the subject of widening the gage on curves should be investigated:

We recommend the appointment of a committee to confer with a committee of the American Railway Master Mechanics' Association for joint report on the necessity of such widening and the amount of widening for wheelbases varying by one foot, and curves varying by one degree, and the formula used determining same, to the end that uniformity of practice may be secured.

It is becoming customary to flange all wheels and the wheelbases are being lengthened. Not all roads are fortunate in having light curvature, consequently it frequently happens that with track spiked to standard gage, the flanges impinge against the inner rail of curves, to the detriment of track and engine power. With a given wheelbase and wheelgage, one may readily figure what widening the ordinate requires, but it is not always known what variation in wheelgage is allowed or how much lateral play in the frame. If there is any uniformity followed by locomotive builders in these respects, it should be generally known; if there is none, it should be urged.

In the discussion, Mr. Kittredge (C. C. C. & St. L. Ry.) stated that some new engines having all the driving wheels flanged are causing trouble on curves of less than 6°, while at frogs they have spread the leads so that cars have been derailed. Mr. Baldwin (Ill. C. Ry.) thought there was a tendency to minimize the importance of uniformity and accuracy in gage of track.

WOODEN BRIDGES AND TRESTLES.

This committee presented in revised form the specifications which were submitted and discussed last year (Engineering News, March 23, 1905), and referred back to the committee for rearrangement. To these were added, however, new specifications for the metallic details used in such structures, and also a recommendation for safe unit stresses, as given below. All these were adopted:

The following allowable unit stresses are recommended, their values being chosen so as to include the allowance for impact:

Stringers.—For long-leaf yellow pine and Pacific Coast red fir: 1,200 lbs. per sq. in. on the extreme fibers in bending, and 120 lbs. per sq. in. for shear parallel to the fibers.

Caps and Sills.—For long-leaf yellow pine and Pacific Coast red fir: 300 lbs. per sq. in. for bearing on the side of the fibers.

For white, post and burr oak: 500 lbs. per sq. in.

Posts and Piles.—For long-leaf yellow pine and Pacific Coast red fir: $1{,}200 - 18 \frac{l}{d}$ lbs. per sq. in. for the average compression on the cross-section, in which "l" is the length and "d" the least diameter of the column, both expressed in inches.

For white, post and burr oak: $1{,}000 - 15 \frac{l}{d}$ lbs. per sq. in.

For Norway pine, cypress and cedar: $800 - 12 \frac{l}{d}$ lbs. per sq. in.

WATER SERVICE.

This committee confined its work entirely to the collection of information from individual railways which have installed water-softening plants, and its report consists of the returns from 30 railways. These roads have from one to 65 plants each, and the average cost of treatment ranges from 1 to 18 cts. per 1,000 gallons; the lowest figure given is 0.5 ct. and the highest 28.5 cts. In most cases the returns report a decrease in boiler repairs, boiler failures on the road, and boiler washing, together with an improvement in the handling of trains.

This closed the proceedings except that a brief verbal report of progress was made by Mr. W. McNab (Grand Trunk Ry.) for the committee on Economics of Railway Location.

OFFICERS.

The officers for the current year are as follows: President, Henry G. Kelley; Vice-Presidents, A. W. Johnston and Walter G. Berg; Treasurer, W. S. Dawley; Secretary, E. H. Fritch. The president's term of office is two years, and this is Mr. Kelley's second year. Mr. E. H. Fritch has been Assistant Secretary for some years, and now takes the place of his brother, Mr. L. C. Fritch, who is in railway service.

A LARGE GAS PLANT is to be established at Chicago to concentrate the manufacture now conducted at eight separate plants of the People's Gas Light & Coke Co. The site extends from 31st to 39th St., and South 40th to South 46th Ave., with an area of about 300 acres. The Chicago River runs across the site, which also has a frontage on the Drainage Canal and the Chicago & Western Indiana R. R., so that with other convenient railway connections there will be ample facilities for receiving coal and shipping coke. The present plants have a total capacity of 40,000,000 cu. ft. of gas per day. The new plant will have units of 10,000,000 cu. ft. capacity and plans are now being made for the first two of these units, which will cost about $5,000,000 each and will be completed in four or five years.

REPORT ON SEWAGE PURIFICATION EXPERIMENTS AT COLUMBUS, O.

To Columbus, O., belongs the honor of having carried out the first large-scale experimental tests of various methods of sewage treatment at all comparable with the numerous municipal sewage experiments in England and the half-dozen municipal water purification experiments in the United States. The Columbus tests extended from Aug. 15, 1904, to Aug. 1, 1905, and the time between the latter date and Nov. 10, 1905, was required to compile the information secured. The methods and results of the tests have just been made public in a report of 499 octavo pages, by Mr. George A. Johnson, Engineer-in-Charge of the Sewage Testing Station, to Mr. Julian Griggs, M. Am. Soc. C. E., Chief Engineer of the Board of Public Service of Columbus, O. (Mr. Griggs was succeeded on March 1, 1906, by Mr. Henry Maetzel.)

All the sewage tested was screened and most of it was passed through a so-called grit chamber. The subsequent treatment included sedimentation at high and low rates, chemical precipitation, septic treatment, and rapid coke filters or strainers, all the foregoing being preliminary or preparatory processes. The final treatment included intermittent sand, contact, and percolating filters (the latter called sprinkling filters in the report). Some of the sand filters were dosed with effluent from a percolating filter and some from a contact filter, the percolating effluent having been settled. The contact filters, for the most part, received settled sewage at high rates of flow through the settling tanks, effluent from coke strainers, and to a more limited extent septic sewage. The percolating filters received effluents from settling and septic tanks.

For convenience, the practical conclusions drawn from the tests will be given before proceeding further. It was recommended by Mr. Johnson, and the recommendation was concurred in by Mr. Griggs, that in the sewage treatment works to be built a preliminary clarification of sewage should be secured by means of septic tanks with a capacity equal to eight hours' flow; that the septic effluent should be treated on percolating or sprinkling filters at a net rate of 2,000,000 gallons per acre per day; and that the effluent from these filters should receive a final clarification by means of sedimentation for two hours. This combined treatment, it was held, would give a non-putrescible effluent of satisfactory appearance, from which about 90% of the bacteria in the raw sewage would have been removed. Plans for carrying out these recommendations have been made, as outlined in a statement of Mr. Griggs published in our issue of Sept. 28, 1905 (p. 313).

GENERAL CONDITIONS.

The population of Columbus, by the census of 1900, was about 125,000, and the present estimated population is 150,000. Probably about 100,000 people are now connected with the sewerage system, which is on the combined plan. About 90,000 of the population connected are on sewers discharging into the Scioto and Olentangy Rivers, which have a combined drainage area of about 1,605 sq. miles. The stream flow below Columbus ranges from 30 cu. ft. to 50,000 cu. ft. per sec. The ordinary minimum flow is given as 30 to 50 cu. ft. per sec., corresponding approximately to a dilution of 0.5 ft. per sec. per 1,000 connected population. On the basis of 4 cu. ft. per sec. per 1,000 population, the minimum stream flow would be able to dispose of the sewage by dilution, without nuisance, of about 10,000 people. The pollution of the Scioto River at and below Columbus already gives rise to objectionable sights and odors, but as the river receives several tributaries within a short distance of the city, the nuisance does not extend far down stream. For the 100 miles of the river between Columbus and its mouth no public water supplies are drawn from the river, and practically none are taken from the Ohio in the distance named, but at the end of the latter distance Cincinnati, O., and Covington and Newport, Ky., draw water from the river for domestic consumption. The problem of sewage disposal at Columbus, therefore, is held to be one of "eliminating nuisances in the immediate vicinity of the city."

Columbus lies in a limestone valley within the limits but near the termination of the glacial drift. At the site purchased for the sewage disposal works coarse sand is found at from 3 to 5 ft. below the surface, but this is "less available for sewage purification than would appear at first sight, owing to the fact that it is situated quite low with reference to ground water level."

A series of reports on sewage purification for Columbus, beginning on May 1, 1898, have been made by Messrs. Julian Griggs, M. Am. Soc. C. E., City Engineer, and later Chief Engineer of the Board of Public Service; John W. Alvord, M. Am. Soc. C. E., of Chicago, and Rudolph Hering, M. Am. Soc. C. E., of New York. Mr. Hering and Mr. George W. Fuller, M. Am. Soc. C. E., were engaged about two years ago as consulting engineers for carrying out the works finally decided upon. A bond issue of $1,200,000 for sewage treatment works, including land, pumping stations and accessories, was authorized by popular vote in November, 1903. Before beginning definite plans for these works, it was decided, upon the advice of Messrs. Griggs, Hering and Fuller, to devote $46,000, or approximately one year's interest on the estimated cost of the improvements, to large-scale experimental tests of various methods of treatment before deciding just what system or combination would be advisable. In January, 1904, Mr. John H. Gregory, M. Am. Soc. C. E., was retained as Principal Assistant Engineer for both improved sewerage and water supply systems. The testing station was designed under his immediate direction. On June 15, 1904, Mr. George A. Johnson reported to Chief Engineer Griggs for the purpose of taking charge of operations at the sewage testing station. The tests were begun about the middle of August, 1904, and from that date until June 30, 1905, about 350,000 gals. of sewage were treated daily by various methods. During the following month some special tests were made. The analyses completed during the tests included 3,270 chemical, 4,356 bacterial, 483 special and 130 sludge examinations. Besides Mr. Johnson, there were employed at the station Mr. A. Elliott Kimberley, as chemist; Mr. William R. Copeland, as bacteriologist, and, in addition, three assistant engineers, two assistant chemists, two assistant bacteriologists, three inspectors, a janitor and for a portion of the time a stenographer. The expenditure on the investigations was divided as follows:

First cost of station, equipment and land	$19,094
Filtering material	349
Laboratory equipment (apparatus and supplies)	3,098
Office furnishings and supplies	1,002
General supplies (including tools, ice, gas)	1,383
Electric current for pumping	1,047
Salaries	17,375
Incidentals	296
Total	$44,004

The balance of $1,996 on hand at the time the report was completed was reserved for the expense of publishing the report.

The ordinary dry weather discharge through the intercepting sewer on which the testing station was built was shown by weir measurements to be 9,100,000 gallons a day, and the maximum daily discharge was 30,500,000 gallons on April 20, 1905, when 0.59-in. of rain fell in a few hours. The maximum capacity of the intercepting sewer is about 40,000,000 gallons a day, so that "at times of average dry weather flows a rainfall of about 0.02-in. per hour over the entire area will cause the main sewers to overflow to the river." The trades wastes reaching the sewers are so porportionately small in volume "that they will doubtless almost lose their identity at the main sewage works."

In collecting samples of sewage for analysis, every eighth day was omitted, thus relieving the analysis and at the same time moving the sampling day forward one day in each week. By this method "no one day was omitted from the sampling schedule twice in succession, but only after the lapse of several weeks." The samples were collected from the screening chamber after screening, at half-hour intervals. They were placed in 4-oz. bottles, which were completely filled, tightly stoppered and placed at once in an ice chest and thus kept at a temperature of about 10° C. About 32 hours after the collection of the first sample the contents of the 48 small bottles were mixed in equal proportions in a large bottle and the chemical analysis begun immediately thereafter. In the bacterial analyses samples were not stored for more than about an hour. The methods of analysis employed, including some new ones, are described in detail in the report.

On the average, the sewage reached the testing station about two hours after its discharge. It usually contained appreciable amounts of dissolved oxygen, except between from about 10 a. m. to 4 p. m., the average for the full day of 24 hours being about 0.5 part per 1,000,000.

Colloidal matters are discussed briefly in the report. They are defined, "from a broad practical standpoint," as "those suspended particles in sewage which cannot be removed by plain subsidence in a reasonable and economical period of time." Such matters are very finely divided, and "appear to exist in a state of pseudo-solution or micro-suspension." They are "a function of both the velocity of sewage flow and the period of subsidence." The colloidal matters in Columbus' sewage, or that portion of the suspended matter not practically removable by subsidence, averaged about 60 parts per 1,000,000.

The fats in the Columbus sewage are low, notwithstanding the hardness of the water and the consequently large amounts of soap which it might be expected would be used; this appears to be due to the fact that cistern water is widely used.

The average chlorine content of the crude sewage is about 65 parts per 1,000,000, as compared with some 50 parts for New England sewages. Examination of water from 18 wells showed chlorine contents of from 9 to 57 parts and an average of 34 parts per 1,000,000. Iron (Fe.) in the crude sewage averaged 16.5 parts per 1,000,000, of which 1.6 parts was dissolved and 14.9 was suspended. The bacterial contents of the crude sewage ranged from 320,000 to 27,000,000, and averaged 3,600,000 per c. c. Ten tests showed B. coli communis ranging from 50,000 to 1,000,000 and averaging 500,000 per c. c. During the tests 90 bacterial organisms were isolated from the sewage, classified and identified.

THE TESTING STATION AND THE TESTS.

A detailed illustrated description of the sewage testing station, prepared by Mr. Gregory, was published in Engineering News for Oct. 20, 1904. Briefly, it may be stated that the station included a pump chamber in the 72-in. intercepting sewer; a 4-in. centrifugal pump; a screening chamber; seven uncovered rectangular tanks about 40 ft. long, 8 ft. wide and 8 ft. deep at the inlet and 9 ft. at the outlet end, for preparatory treatment by sedimentation, chemical precipitation and the septic process; two coke strainers; six contact filters; six sprinkling or percolating filters; 21 intermittent sand filters; and a laboratory and office building. The latter was 32 × 64 ft. in plan.

SCREENS.—Before the sewage went to the pump it passed through a 0.5-in. clear mesh screen. The screening chamber beyond the pump was provided with two movable vertical screens, the first having a clear 0.5-in. and the second 0.375-in. mesh. The screens removed about 300 lbs., or 0.17 cu. yd., of wet screenings per 1,000,000 gallons of sewage, on the average. All the sewage tested was screened.

TANKS FOR PRELIMINARY TREATMENT.—The seven tanks mentioned above were filled within 12 ins. of the top, thus giving a capacity of 17,600 gals. each, without any allowance for sludge. None of the tanks had baffles, except that the inlet pipe to each terminated in a box 4 ft. square, placed 5 ft. above the bottom of the tank, for the purpose of lessening sludge disturbances and giving a more uniform flow. One of these tanks was designated a grit chamber; the others were used for sedimentation at high and low velocities, chemical treatment and for the septic process.

GRIT CHAMBER.—The so-called grit chamber

was really a settling tank operating at a high velocity of flow. As such, it gave results so similar to the other settling tanks that in April, 1905, it was reconstructed in smaller size (39.5 × 5 ft. in plan and 3 ft. in depth). The original grit chamber was operated from Aug. 16 to Feb. 17 at a rate of flow 2.61 mm. per sec., giving a time of passage of 1.3 hours. From Feb. 18 to June 30 the remodeled and relatively narrower tank was operated with a velocity flow of 11.39 mm. per sec., giving a detention in the tank of 0.29-hour. The principal reason given by Mr. Johnson for remodeling the grit chamber was that the sludge deposit in the larger so-called grit chamber differed but slightly from that in the low velocity settling tanks. By increasing the velocity of flow it was hoped that the deposition of mineral matters would be much increased as compared with that of organic particles. The result "was a diminution in the subsiding efficiency, but no material change in the relative removal of volatile and mineral suspended matter."

All of the sewage treated passed through one or the other of the grit chambers before being given the other preparatory treatment, except that the closed septic tank was supplied with sewage which had been screened only.

SEDIMENTATION TANKS.—The sedimentation tanks, operated at low rates, had velocities of flow averaging 0.42 and 0.56 mm. per sec., or 4.9 and 6.7 ft. per hr., respectively, giving passage periods of 8 and 6 hours.

SEPTIC TANKS.—In the two septic tanks, of regulation size (17,000 cu. ft. holding capacity), the passage periods were 8 and 16 hours for about the first half of the period and 16 and 24 hours, respectively, thereafter. These figures correspond to velocity flows of 0.42 and 0.21 mm. per sec., or 5 and 2.5 ft. per hr., and 0.21 and 0.14 mm. per sec., or 2.5 and 1.7 ft. per hr., respectively. A smaller tank, circular in form and provided with baffle boards, was operated with a 4-hour passage period. A closed (air-tight) boiler iron tank 15 ft. long by 6 ft. diameter, with a mean effective depth of 5 ft. and a holding capacity of 7,200 gals., was operated for about four months, with a passage period of 8 hours. A pipe leading to a gas meter was connected with this tank. The percentage of gas evolved to volume of sewage treated, by months (1905), was as follows: March, 3.1; April, 1.5; May, 4.4; June, 7.0. The average temperature of the applied sewage during these months was 51, 58, 61, 66° F. It was concluded that "open tanks are quite as satisfactory for the treatment of the local sewage as those provided with covers." On none of the septic tanks did there appear regularly a well-defined scum formation.

CHEMICAL PRECIPITATION.—In the chemical precipitation tanks both sulphate of alumina and a combination of copperas and lime were used, the average passage period being eight hours and the velocity flow about 0.42 mm. per sec. The report states that the chemical precipitation tests added nothing to the knowledge of the subject, so far as the coarser matter was concerned, but were valuable

in showing that the use of chemical precipitants to remove fine suspended matter, or what may be called colloids, is not a satisfactory or practical procedure when the chemicals are applied directly to the sewage of the grit chamber. In other words . . . if the fine submicroscopic particles were to be removed prior to filtration it is first necessary to deposit the coarser suspended material and then coagulate the fine particles along the same line as was first developed in the treatment of the muddy Ohio River water at Louisville.

Another point shown by the chemical precipitation tests was that an eight-hour period was too short because it allowed some of the hydrate to pass onto the sand filters, thus clogging them seriously. This

showed that this aspect of the problem was quite similar to coagulation of muddy river water by sand filtration. Such investigations at Cincinnati and New Orleans demonstrated that the sedimentation period following the application of a coagulant should be at least 24 hours.

COKE STRAINERS.—The coke strainers (for preliminary treatment) were in the form of circular tanks; one had an area of 0.001 and the other 0.003-acre. Each was underdrained with layers of graded gravel, making 12 ins. in all, and above the gravel was placed 27 ins. of Kanawha coke, ranging in size from 0.5 to 0.125-in. The coke was screened and was comparatively hard and durable. The strainers were put in use on Aug. 17, 1904, and continued in use until Feb. 1 and May 31, 1905, respectively, during which periods one had tested a total volume of sewage corresponding to 457,460,000 gals., and the other 421,000,000 gals., per acre; the first had been scraped three times and the second one twice. The coke in each filter was found matted together from top to bottom in a thick gelatinous layer. It was concluded that the most practicable method of disposal of clogged material from coke strainers would be to mix it with fuel and burn it beneath boilers. Experiments showed that washing the clogged coke was impracticable, and that drying it (for fuel) took too long and gave rise to an exceedingly offensive odor, noticeable at a distance of 200 yds. The coke cost $1.35 per cu. yd.

INTERMITTENT SAND FILTERS.—The sand filters were also placed in circular tanks; each filter had an area of 0.001-acre. No underdrains were provided except just above and around the mouth of the outlet pipes. The sand in these filters was 3 ft. deep and had effective size of from 0.20 to 0.29 mm. and an average of 0.24 mm.; the uniformity coefficient ranged from 2.1 to 2.6 and averaged 2.5.

The operation of the sand filters followed, in general, the usual lines, and the results need little comment further than to state that they emphasized anew the fact "that the rate of treatment possible in filters of fine-grain material is chiefly controlled by the amount and character of the suspended matter contained in the applied sewage." In addition it may be noted that while the evidence is not complete it appears that sand filters could satisfactorily treat the *settled* effluents from percolating or sprinkling filters at rates ranging from 2,000,000 to 3,000,000 gals. per acre per day; and that with contact filters *as efficient* as were the sprinkling filters at Columbus the effluent from contact filters might be treated on sand filters at as high a rate as on sprinkling filters—but such contact effluents as were yielded at Columbus could not be "successfully sand-filtered at rates exceeding about 500,000 gals. per acre daily." Further treatment of settled effluents from percolating filters is not considered necessary at Columbus.

CONTACT FILTERS.—The contact filters, also circular in shape, had areas of 0.001 and 0.003-acre, two of the smaller ones serving as secondary filters. For underdrains all of the contact filters first had 4 ins. of gravel from 3 to 1.75 ins. in size, and 2 ins. of gravel from 1.75 to 0.625-in. These underdrains not proving satisfactory, they were replaced late in April, 1905, by 6 ins. of 4 to 5-in. stone, 4 ins. of 2 to 3-in. stone, and 2 ins. of 1 to 2-in. stone, and above this new material the old material was placed. The filtering material in each of the contact filters was 5 ft. in depth, of either broken limestone from local quarries costing $1.20 per cu. yd, at the station, or else crushed coke. The size of the material in each of the filters, and the percentage of voids, were as follows:

Filter.	Material.	Size, Ins.	Per cent of voids.
Primary A	Limestone	1 to 0.25	43.1
Primary B	Limestone	2 to 0.25	47.0
Primary C	Coke	1.5 to 0.5	54.0
Primary D	Coke	1.5 to 0.5	54.0
Secondary A	Coke	1.25 to 0.25	49.6
Secondary B	Coke	1.5 to 0.5	54.3

As to the operation of the contact beds attention may be called, first of all, to the fact that the sewage which they received, for the most part, appears to have had less preliminary treatment than that given to the sewage applied to the other filters; and that comparatively little septic tank effluent seems to have been applied to the contact beds. So far as could be judged from the limited period of observations, the broken limestone and coke used for the contact beds would suffer but little disintegration in practical use. It also appeared that the two materials would be equally efficient, under the conditions prevailing at Columbus. The advantage of septic tank effluent over effluent from the so-called grit chambers or from sedimentation was believed to be wholly due to the relatively lower amount of suspended matter in the septic effluent, rather than to any "material change in the dissolved organic constituents of the applied sewage." In stronger sewages this might be otherwise.

Quick filling and emptying of contact beds and a long period of rest seemed to be the most important features in the contact bed cycles. An hour of contact seemed to be ample. The report states that "conditions obtaining during the tests were such that denitrifying actions outweighed the normal aerobic agencies at work in the contact filters." It appeared that with the secondary contact filters could be operated at double the rates of primary contact beds, and that double contact would make permissible net rates per acre some 20% in excess of what could be practiced satisfactorily with single contact on the same gross area of land.

As to the theory of action of contact filters the report states that when such filters came into use they "were considered as extensions" of intermittent sand filters and their work "was ascribed to bacterial activity." Physical and mechanical actions are now recognized, precedence being given "to the absorption power of organic colloidal matter and the properties of adhesion and gravity, respectively, rather than to the action of bacteria alone." When a contact filter is standing full the "gelatinous coating of organic colloidal matters left upon the particles of filtering material, from previous flooding, removes by adsorption a certain proportion of dissolved putrescible matters." On emptying, the admitted air results in bacterial action and the consequent formation of nitrates, nitrites and carbon dioxide. The oxygen soon being oxidized, anaerobic activities begin, complex putrescible matters are broken down and some reduction is effected of the previously formed oxidized nitrogen compounds. Under subsequent fillings the deposition and adsorption of organic matter is resumed, and the denitrifying bacteria break down a considerable portion of putrescible matter. Non-putrescible effluents from contact filters seem to depend upon an "excess of nitrified nitrogen over and above the amount required for the biological exidation of the putrescible compounds which escaped oxidation within the filter."

The reduction in the voids of the primary contact filters ranged from 15.2 to 32%, the higher rate being for Bed A, which rested only 28 out of 317 days, as compared with 57 out of 317 days for one of the beds and 36 out of 313 for the other two. The loss of voids in the two secondary beds was 15.9% for Bed A and 14.9 for Bed B. The two beds were operated 300 and 313 days, respectively, with a total resting period of 45 days in each case. It will be understood, of course, that other factors than resting had to do with the clogging of the beds.

SPRINKLING FILTERS.—Of the six percolating or sprinkling filters one had an area of 0.003-acre, four of 0.001-acre, and one was slightly larger than the latter. The depth of the filtering material in each percolating filter was 5 ft., except that one of the filters of smallest area had a depth of only 1.8 ft. With one exception, the size of the material (broken limestone) in these filters ranged from 2 to 0.5-in., the size of the material in one case ranging from 1.75 to 0.5-in. After trying perforated troughs of galvanized iron, which sagged, and perforated iron pipes with pet-cocks, which clogged for several times daily, and perforated cones in place of some of the pet-cocks, stationary nozzles were finally adopted for sprinkling the sewage over the percolating filters. These nozzles were under a 4-ft. head and worked well. They consisted of a hollow 60° cone, 2 ins. in outside diameter at the base, over which was placed a similar cone of 2 ins. inside diameter. The outside cone was truncated 1.2 ins. from the base. Through the inner cone were passed eight 0.125-in. brass pipes, which discharged into the adjustable space between the two cones, whereupon the whole flow was sprayed from the truncated top of the nozzle.

The percolating or sprinkling filters really differed from the contact filters more in the method of applying the sewage than in the size and ar-

rangement of the filtering material. The small size of the percolating material and the poor facilities for drainage, compared with British practice, doubtless account, in large part, for it being "found most satisfactory to operate the [percolating] filters only about half the time." It is also worthy of note that even after the sprinkling devices had been improved by changing to nozzles only 70% of the total filtering area was wetted by the sprinklers, barring periods when this was exceeded by wind action. An improved nozzle, "tested in August, 1905, under the general direction of Mr. Gregory," sprayed "about 77% of the area tributary to each nozzle." It worked under a head of 3 to 5 ft., and had a 9-16-in. discharge nozzle.

The percentage of voids in five of the percolating filters ranged from 42 to 43.1%, and the loss in voids was from 2.3 to 9.7%, after from 230 to 313 days' operation. Directly beneath the sprinklers the filtering material for a depth of 2.5 ft. showed "a black, sticky deposit," and the lower portions of the underdraining material "were badly matted together," with a "very gelatinous, strongly adhesive" deposit, of "a deep black color and a foul odor." This portion of the underdraining material, on being exposed, showed "pools of liquid." The foul deposit "contained considerable quantities of organic matter and some sulphide of iron."

Severe winter weather gave rise to "no apparent deterioration in the efficiency of the sprinkling filters." No difficulty with the sprinkling nozzles was experienced, not even with an average air temperature of —1° F. for 24 hours. (The minimum average sewage temperature was 39° F.) Ice rings formed around the outer edges of the percolating filters to a width of 3 ft. and a depth of 0.3-ft. The effluent, during the coldest weather, averaged only about 3° F. lower than the temperature of the applied sewage.

The action within percolating filters is described in the report as in many respects similar to that in contact filters, but it is stated that the bacterial effects are normally aerobic, and these and the physical actions within the filter, particularly the drying of the deposits on the filtering material and the flushing effects on starting up the sprays after a period of rest, result in large quantities of suspended matters in the effluent, "to the temporary detriment of the appearance of the effluent, but to the ultimate benefit of the filter."

Mr. Johnson's review of the tests and his summary of the conclusions drawn therefrom will be given in a later issue.

NOTES FROM THE ENGINEERING SCHOOLS.

HARVARD UNIVERSITY.—A Graduate School of Applied Science has been established, to begin with the academic year 1906-7. For the present degrees will be granted in engineering (mechanical, electrical, and civil), mining, metallurgy, architecture, landscape architecture, forestry, applied chemistry, applied biology, and applied geology. A bachelor's degree in arts or science, or its equivalent, will be required for admission to this school. Another change at Harvard, to begin next fall, will be the granting of the degree of Bachelor of Science, *without* designation of any field of study, by Harvard College, as distinguished from the Graduate School, just mentioned, and also from the Lawrence Scientific School. The latter will continue "as long as it shall seem best" to grant the degree of Bachelor of Science, *with* designation of the field of study, to students completing any one of the four courses in "civil and topographical engineering, mechanical engineering, electrical engineering, mining and metallurgy, architecture, landscape architecture, forestry, chemistry, geology, biology, anatomy and physiology, for teachers of science, and general science." The new scheme contemplates five or more years' instruction in Harvard College and in the Graduate School of Applied Science, jointly: that is, "choosing undergraduate studies freely" in Harvard College for 3, 3½ or 4 years for a general degree of B. S., and then taking two or more years in the Graduate School of Applied Science in order to secure a professional degree. At the same time, the alternative of obtaining a professional degree in the Lawrence Scientific School remains open, for the present.

A COAL MINE EXPLOSION in the Century mine, near Philipps, W. Va., on March 22, killed thirty miners. The accident was caused by dust.

A BAD RAILWAY ACCIDENT occurred on the Denver & Rio Grande Ry., near Portland, Col., on Friday, March 16. An east-bound passenger train collided with a west-bound passenger train, both being fast through trains, and the wreckage burned up. Twenty-five to thirty-five persons were killed, either in the collision or in the fire that followed. The trains were to have passed at Florence, but the west-bound train being delayed, orders were issued to pass 12 miles east of Florence. The east-bound train received this last order, but the operator at Swallows, Col., who was to deliver the order to the west-bound train, was asleep and failed to do so. The trains collided on a curve, in a heavy blizzard, and had only a few seconds warning before they met. The operator responsible for the accident was working overtime, taking the place of the man who should have relieved him, but this arrangement was made without the knowledge of the dispatcher, it is said.

FIRE IN THE PENNSYLVANIA R. R. TUNNEL under the East River at New York City on March 19 resulted in the death of one man and the stoppage of all work for several hours. The tunnel is being driven by shield under air pressure and begins at the bottom of a 115-ft. shaft at 33d St. and First Ave. There is an air lock at the foot of the shaft, another further on in the bore and a working shield at the head of the advance. The air pressure in the working compartment is 36 lbs. and in the compartment between the two air locks it is 20 lbs. The fire occurred in the working compartment and is thought to have started in a pile of hay used for calking leaks. It took place in the interval between changing of shifts and the only man in the working compartment was the watchman who was overcome by smoke and so badly burned that he died. Two boxes of dynamite were in the working compartment and a courageous act was performed by one of the foremen, George Barr, who alone passed through the two air locks and into the burning compartment and carried the boxes to the surface one at a time, falling exhausted at the completion of the last trip. Later Acting Chief George Fox, of Engine Crew 21, New York Fire Department, passed through the two locks and rescued the body of the foreman whose death was noted above. It being impossible to get hose to the fire it was allowed to burn itself out, which it did in about seven hours. The tunnel is being constructed by Pierson & Co. as contractors.

A NEW PASSENGER TERMINAL STATION at Chicago is again under discussion and it is now reported that the Union Station, on Canal St., will be rebuilt. The station building will be on the east side of Canal St. (as at present), between Madison and Adams Sts. But the trainshed and yards, instead of being on the east side of the building, will be west of Canal St., communication being afforded by subways under the street. The present station is used by the Pennsylvania Lines (both the Pittsburg, Fort Wayne & Chicago Ry. and the Pittsburg, Cincinnati, Chicago & St. Louis Ry.), the Chicago & Alton Ry., Chicago, Milwaukee & St. Paul Ry., and the Chicago, Burlington & Quincy Ry. Its average daily traffic in 1905 was 246 trains, with 1,448 cars, but none of these roads has any extensive suburban service. It has been suggested that the Chicago & Northwestern Ry. should use the new station, but this road has an immense suburban traffic and operates 242 trains per day at its Wells St. station, so that it prefers to retain an independent terminal, and is preparing plans for an increase in its facilities. These will include a four-track bascule bridge over the Chicago River to replace the present double track swing bridge which throttles the traffic just at the entrance to the passenger station.

SINGLE-PHASE SUBURBAN LINES are being built by the Milwaukee Electric Railway & Light Co., of Milwaukee, Wis., whose network includes the extensive street railway system of Milwaukee and a number of suburban lines, all direct current. One of the existing suburban lines, running from Milwaukee about 15 miles to Waukesha, is to be extended 20 miles to Oconomowoc. Another line, now running to Hales' Corners, will be extended to Mukwonago, 16 miles. These two extensions are to be alternating-current single-phase lines, using a trolley-pressure of 3,300 volts. The motors will be designed to run also on direct-current city lines (550-volt). Ten cars are being built for these extensions. General Electric single-phase motors have been ordered for them. Each car will carry four 75-HP. motors (GEA-605), which are four-pole motors running at about 140 r. p. m. maximum speed. In order to make it possible to run two-car trains, the cars are equipped with Sprague-G.E. multiple-unit control, modified for operation with either direct or alternating current. The control connections are so designed as to avoid breaks in the current during the acceleration, in order to eliminate jerking. The trolley line will be supplied by a 33,000-volt transmission line feeding transformer stations. The two extensions are planned to be in service by the end of 1906.

THE CARNEGIE STEEL TIES on the Bessemer & Lake Erie Ry. were described and illustrated in our issue of Aug. 21, 1905, and the annual report of the General Manager, Mr. E. H. Utley, states that the purchase of 25,000 ties was made after a thorough test on half a mile of track, the ties of which were laid in the fall of 1904, and which held up under the heavy traffic of 1905 with satisfactory results. For the new work authorized for 1906, as well as for maintenance, 80,000 additional steel ties have been purchased. These make 105,000 in all, weighing 9,550 tons, and sufficient for 42 miles of track.

PERSONALS.

Robert Hawxhurst, Jr., Assoc. M. Am. Soc. C. E., is at present in China, on professional business.

Mr. O. M. Laing, Assistant Superintendent of the Central New England Railway Co., will have headquarters at Poughkeepsie, N. Y.

Mr. Charles H. Copley, General Manager of the Bellows Falls-Saxton River Electric Railway, of Bellows Falls, Vt., has resigned, taking effect April 15.

Mr. A. Q. Campbell has been appointed Engineer of Track and Structures of the Atlantic & Birmingham Construction Co., with headquarters at Oglethorpe, Ga.

Mr. H. O. Buck, Assistant Engineer, has resigned his position with the Central New England Railway Co., and is succeeded by Mr. E. R. Clarke, with headquarters at Hartford, Conn.

Mr. J. T. Johnson, formerly General Superintendent of Transportation of the Central of Georgia Ry., has been appointed General Superintendent of the same company, with headquarters at Savannah, Ga.

Mr. C. L. Brown has been appointed Superintendent of the Chicago, Rock Island & Pacific Ry. at Fairbury, Neb., to succeed Mr. C. J. Wilson, who has been transferred to Esterville, Ia., to succeed Mr. Brown.

Mr. James L. Armstrong has been appointed Chief Engineer of the Cairo & Thebes Railroad Co., which embraces the terminals and yards in Cairo, Ill., and the bridge at Thebes, Ill., with headquarters in Cairo, Ill.

Mr. W. R. McDougall, of Birmingham, Ala., has been appointed General Superintendent of the Atlanta & Birmingham Railway, with headquarters in Atlanta. Mr. McDougall was formerly with the Southern Railway Co.

Mr. E. C. Murphy, Engineer in the Hydrographic Branch of the U. S. Geological Survey, has been instructed to proceed to Texas to inspect the gaging stations maintained in that state by the Geological Survey.

Mr. W. J. Towne has been promoted from Division Engineer of the Chicago & North Western Railway Co. to a position in chief engineer's office in charge of permanent work, and other special work which may be assigned to him.

Mr. Albert O. Smith, Jun. Am. Soc. C. E., has opened an office at Greenport, N. Y., where he will practice his profession as a civil engineer. In the issue of March 15, Mr. Smith's address was given at Greenpoint, N. Y., instead of Greenport.

The office of Alex. Bonnyman, Chief Engineer of the Atlantic & Birmingham Ry. Co. and the Atlantic & Birmingham Construction Co. has been removed to 24 East Alabama St., Atlanta, Ga., where all correspondence should be addressed.

Mr. A. E. Hess, Principal Assistant Engineer on Surveys for the Pensacola, Alabama & Western R. R., at Selma, Ala., has resigned and has been appointed Chief Engineer of the Augusta Construction Co., with headquarters at Augusta, Ga.

Mr. Roland Allwork, Inspecting Engineer Isthmian Canal Affairs sailed on Saturday, March 24, on the S. S. "Allianca" for the Isthmus, where he has been ordered temporarily. During his absence Mr. W. A. E. Doying will serve as Acting Inspecting Engineer.

Mr. A. W. Grosvenor, Assistant Engineer of Maintenance of the Western division, Northwest system of the Pennsylvania lines, has been transferred to New York with title of Fourth Engineer of the Pennsylvania R. R. Co.'s terminal improvements at that place.

Mr. A. E. Welby, General Superintendent of the Rio Grande Western Railway, has been appointed General Superintendent of the Denver & Rio Grande R. R. No successor will be appointed to Mr. Welby at Salt Lake,

his duties being merely broadened to give him control of the entire Denver & Rio Grande and Rio Grande Western systems.

At the recent annual meeting of the Wellman-Seaver-Morgan Company, of Cleveland, Ohio, the office of General Manager, which has been vacant since the death last June of Mr. Charles H. Wellman, was filled by the election of Mr. S. H. Pitkin, whose present title will be First Vice-President and General Manager. No other changes were made in the officers of the company.

Mr. W. W. Ewing has resigned the position of Engineer and Treasurer of the Thos. J. Brady Co., of New York City and has accepted an appointment as Engineer in the Construction Department of Westinghouse, Church, Kerr & Co., taking effect on April 1. Mr. Ewing was for some years Chief Engineer of the New York Building Department and later built a large cement plant in Spain.

Mr. C. M. Johnston, Superintendent with S. Pearson & Son, Ltd., on the Pennsylvania, New York & Long Island R. R., on the East River tunnel contract, has resigned to accept the position of Superintendent with the Eyre Construction Co., of Philadelphia. Mr. Johnston will superintend the construction of two stone arch bridges over the Juniata River at Mt. Union and Ryde, Pa., for the Pennsylvania Railroad Company's change of line on the Middle Division.

Mr. A. E. Winchester, Superintendent of the municipal electric light plant at South Norwalk, Conn., has been granted leave of absence by the City Council. He will be a member of the special committee of the National Civic Federation on municipal ownership to visit Europe to inspect several plants. He sailed on the Cunarder "Lucania" on March 17 for London, to join the other members of the special commission, whose work is expected to last until August 1.

Messrs. John C. Kafer, Asa M. Mattice and Benjamin H. Warren have formed a partnership and established an office at 60 Wall St., New York City, where they will practice as consulting engineers. Mr. Kafer was Vice-President and General Manager of the late Morgan Iron Works, New York, 1888 to 1900, and Superintendent of the Quintard Iron Works, New York, 1901 to 1904. Mr. Mattice was in the Engineer Corps of the United States Navy, from 1874 to 1889. He acted as the principal assistant to Mr. E. D. Leavitt, Consulting Engineer, from 1889 to 1900. Mr. Mattice was Chief Engineer of the Westinghouse Electric & Manufacturing Co., and later its Consulting Engineer and Chief Engineer of the Westinghouse Machine Co., 1900 to 1904 and Chief Engineer of the Allis-Chalmers Company from 1904 to 1905. Mr. Warren is also a graduate from the United States Naval Academy and was Mechanical Engineer and Superintendent of the Hancock Inspirator Company from 1878 to 1880. He was Assistant General Manager and afterward Second Vice-President of the Westinghouse Electric and Manufacturing Co., from 1896 to 1902, and President of the Allis-Chalmers Co., from 1904 to 1905.

Obituary.

H. G. Coykendall, a veteran railroad contractor, died on March 22 at St. Mary's Hospital in Rochester, Minn.

William R. Hattersley, Secretary and Treasurer of the Hattersley Construction Co., of Toledo, Ohio, died at his residence, 539 Winthrop St., in that city, on March 17. Mr. Hattersley was 30 years of age.

F. L. Dever died at El Paso, Texas, on March 3. Mr. Dever was U. S. Assistant Engineer at Duluth, Minn., and had local charge of dredging in Duluth-Superior Harbor. He was in charge of the breakwater construction at Marquette, Mich., 1903-1904, and resigned on account of ill health in 1905. He is survived by a widow and two children.

Duncan Wirgman Hemming, M. Am. Soc. C. E., died on March 22 at his home, 502 West 143d St., New York City, of cancer of the stomach. He was born in Drummondville, Quebec, and was educated at Bishops' College, in Canada, and after abandoning the study of law for civil engineering, his first position was a rodman on the Canadian Pacific Railway. In 1893 Mr. Hemming became a locating engineer on the New York, New England & Northern Railway, and in 1894, he was made engineer for the Metropolitan Street Railway, in the construction of the Lexington Ave. cable line. He helped to lay gas mains in New York City, and was identified with the Speedway construction. He was the engineer in charge of laying twelve miles of pneumatic tubes from the New York General Post Office to sub-stations, and later helped to build the piers for the Manhattan Bridge, the 145th Street Bridge and the Riverside Drive extension. Mr. Hemming was 45 years of age. He was a mining foreman and instrument man for Ryan & McDonald on the Baltimore & Ohio Belt Line tunnel in Baltimore and progressed rapidly until he became Consulting Engineer for the firm. For the last few years he has been employed as Chief Engineer by Wm. Bradley & Sons, Contractors, of New York, on the Jerome Park Reservoir work, the New York Subway and the New York snow removal contract. He was married in 1896 to Miss Florence Krete of Baltimore, who survives him.

August von Borries, who for years was a leading spirit in the advancement of the steam locomotive in Germany, died on Feb. 14, 1906, at the age of 54. He was born on Jan. 27, 1852. After a thorough technical education he was in minor railway service for several years, and in 1879 entered the mechanical department of the government railways. In 1888 he became chief of the mechanical engineering department of the Hannover railways, with which he remained for 14 years, taking a seat in the directorate of the division in 1894. In 1902 he was called to the Berlin Technical College as Professor of Locomotive Machinery, but in 1905 he was compelled to leave this work by illness which later resulted in his death. His chief work was connected with the introduction of steam compounding in locomotives. As early as 1880 he suggested an improved construction of compound locomotives, which was adopted in the same year for the first two compounds of the Hannover railways. In 1884 the government granted him a prize of $250 for his work in compound locomotives, and his labors were largely responsible for a ministerial decree in 1895 that all passenger and through freight locomotives thereafter built be compounds. In 1892 the German Railway Association gave him first prize, the sum of $1,000, for the development of compound locomotives. Von Borries was a close student of foreign locomotive practice, particularly that of the United States. He visited America in 1891 and 1893, and on the latter occasion was member of the jury of awards at the Chicago World's Fair; he was similarly honored at Paris in 1900. During his professorship at Berlin he set about the construction of a locomotive testing plant, the first in Germany, but he did not live to see its completion. In recent years he gave special attention to four-cylinder compounding, for which he devised an improved valve-gear, to the use of superheated steam in locomotives, and to railway motor cars, which latter he included in his courses at the Berlin Technical College.

ENGINEERING SOCIETIES.

COMING MEETINGS.

IOWA ELECTRICAL ASSOCIATION.
April 18 to 20. Annual meeting at Des Moines, Ia. Secy., Geo. S. Carson, Iowa City, Ia.

AMERICAN RAILWAY ASSOCIATION.
April 25. Semi-annual meeting at Chicago, Ill. Secy., W. F. Allen, 24 Park Place, New York City.

AMERICAN ELECTROCHEMICAL SOCIETY.
May 1 to 3. Annual meeting at Ithaca, N. Y. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
May 1 to 4. Semi-annual meeting at Chattanooga, Tenn. Secy., F. R. Hutton, 12 West 31st St., New York City.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 5. Annual meeting at Atlantic City, N. J. Secy., W. C. L. Eglin, 136 Liberty St., New York, N. Y.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
June 5 to 7. Annual convention at Cleveland, O. Secy., Richard Moldenke, Watchung, N. J.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 18. Annual convention at Atlantic City, N. J. Secy., J. W. Taylor, Old Colony Bldg., Chicago, Ill.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 26. Annual convention at Thousand Islands, N. Y. Secy., Chas. Warren Hunt, 220 West 57th St., New York, N. Y.

WESTERN RAILWAY CLUB.—Three subjects for discussion were assigned for the regular meeting held at the Auditorium Hotel, Chicago, March 20. The first of these related to the rules of interchange governing the loading of long material on freight cars. The Pennsylvania R. R. has argued very strongly that there should be some limitation to the height of load carried on the tops of high-sided gondola cars, and the rules of this railway and the Master Car Builders' Association on this point are as follows:

Pennsylvania R. R.—For loads carried on one bearing piece per car, located at or near center of truck, or on top of sides of gondola cars, the weight of lading must not exceed half the marked capacity of the car; for loads on top of sides of gondola cars the distance from top of rail to center of load, measured at bearing pieces, must not exceed 9 ft. 3 ins.

M. C. B. Association.—For loads carried on one bearing piece per car, located at or near center of truck or on top sides of high-sided gondola cars, the weight of lading must not exceed half the marked capacity of the car.

The two other subjects for discussion related to locomotive repair work, and were intended to bring out the opinions of members on matters which are being investigated by two committees of the American Railway Master Mechanics' Association. The first was in regard to the establishment of a uniform method of classifying locomotive repairs in railway shops in order that reliable comparisons can be made as to the efficiency of various shops on any one railway or between shops on different railways. In a written discussion, Mr. H. H. Vaughan (Canadian Pacific Ry.) suggested that the classification should distinguish between running repairs and shop repairs. Mr. P. H. Peck (C. & W. Ind. Ry.) that with a proper system of records some comparisons of practical value could be made between the shops of any one road, but he doubted whether such comparisons could be made between the shops of different roads. Other members doubted even the practicability of the first class of comparisons, owing to the varying conditions as to operation, shops and locomotive repairing on large railway systems.

The second phase of locomotive repair work discussed was that of the best practice in doing running repair work in the roundhouse; this included the work of firemen, enginemen, road foremen of engines and inspectors; also the proper equipment of hand tools and machine tools for this roundhouse work. A report made by Mr. H. T. Bentley, Assistant Superintendent of Motive Power of the Chicago & Northwestern Ry., treated the matter on the basis of a roundhouse having 50 stalls, but in the discussion it was pointed out that this did not represent average conditions, and that it would be more appropriate and of greater value to take a 15-stall roundhouse as a basis.

WESTERN SOCIETY OF ENGINEERS.—At the meeting held at the society's rooms, in Chicago, on March 23, Mr. W. T. Curtis presented a paper on "A New Method of Calculating Bridge Stresses by Means of End Shears," with a table designed to facilitate the computation of stresses in bridges by means of assumed wheel loads. We expect to publish this paper in full in a later issue. It was shown that the maximum pier reaction is not only useful in determining pier and tower loading, but is a direct factor in the calculation of the maximum moment based upon the following proposition which was originated by Mr. J. Gibson:

The moment at any point in a span is equal to the moment produced by the application at that point of an imaginary load equal to the pier reaction of two imaginary spans joining at the point in question, and of lengths equal to the right and left hand portions of the span, measured from the point. It follows that this moment is a maximum when the pier or panel concentration of the two imaginary spans is a maximum.

The second paper of the evening was by Prof. W. K. Hatt, on "Physical and Mechanical Characteristics of Structural Timber." He showed the different properties of timber required for various manufactures and industrial processes, and described numerous tests of different varieties of timber. The effect of steaming as a preliminary to preservative treatment was also discussed, and it was shown that while this effect was small up to a pressure of 20 lbs., higher pressures reduced the strength very materially.

RAILWAY SIGNALING CLUB.—At the meeting held at the Great Northern Hotel, Chicago, on March 19, the proposed new constitution was discussed at some length, and its adoption was recommended; action upon this will be taken at the next annual meeting. Three papers were presented and discussed, as follows: "The Charging of Storage Batteries from Alternating-Current Circuits," by F. B. Corey (General Electric Co.); "The Substitution of Track Circuits for Detector Bars," by W. N. Spangler (W. J. & S. Ry.), and "The Power Distant Signal," by W. A. D. Short (Ill. Cen. Ry.).

Signal Poles.—The report of the Committee on Standard Specifications for Mechanical Interlocking was presented by Mr. Geo. S. Pflasterer (N., C. & St. L. Ry.), and the following specifications for iron signal poles with concrete foundations were adopted:

All straight signal poles must be of lap welded iron steam pipe in sections of 4, 5 and 6 ins., with sweiged joints, 18 ins. in length and inserted 16 ins. The swedged ends must be hammer-calked. Total weight for 32-ft. poles set in concrete must be not less than 526 lbs.; for 38-foot poles, 639 lbs. One-arm poles in base castings must be 26 ft. long and weigh not less than 413 lbs.; two-arm poles, 32 ft. long and weigh not less than 526 lbs. These weights apply to poles without fittings.

Weights and lengths of pipe to be as follows: 4-in. sections, 9 ft. 4 ins. long, weighing 10.665 lbs. per ft.; 5-in. sections, 11 ft. 4 ins. long, weighing 14.502 lbs. per ft.; 6-in. sections, for poles in ground; one-arm, 14 ft. in length, two-arm, 20 ft., weighing 18.762 lbs. per ft.; 6-in. sections for one-arm poles in base castings, 8 ft. in length; 6-in. sections for two-arm poles in base castings, 14 ft.

Poles which are set in the ground must be reinforced by concrete not less than 12 × 12 ins., 6 ft. deep, and pole filled with cement mortar to 2 ins. above the concrete, and with a ¾-in. drain hole through wall of the pipe at top of filling. Concrete foundation for base castings must have top 30 × 30 ins. and be not less than 5 ft. deep, with 1-in. slope per foot on side walls. Anchor bolts to be 1 in. diameter and extended to not more than 12 ins. from bottom of foundation.

WASHINGTON SOCIETY OF ENGINEERS.—The regular meeting of the Washington Society of Engineers was held at Hubbard Hall March 21. The subject for discussion was wireless telegraphy. L. D. Bliss, Secretary of the society, spoke on wireless telegraphy from a historical standpoint, discussing the various stages of communicating without wires from the signal fires of the ancients to the instruments of the modern wireless companies. He explained at length the different kinds of transmitters and receivers conducting several experiments to show the operation of the machines. R. M. Klein, of the Bureau of Equipment, Navy Department, presented some notes on the general theory of wireless as obtained by him in his work at the department. Lieut.-Commander S. S. Robison, U. S. N., discussed the value of the wireless communication as applied to vessels, and especially to the vessels of the United States Navy. Lieut.-Commander L. A. Kaiser and Maj. Edgar Russell also spoke on the subject.